# An Evaluation of a Proposed Change in Operational Protocol for Structure S-25B to Enhance Flood Management in the C-4 Basin

by

*Mark M. Wilsnack*

*Kenneth D. Konyha*

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**Hydrologic Systems Modeling Division**
**Water Supply Department**

## Executive Summary

In an effort to reduce the potential for frequent flooding within the town of Sweetwater, local officials have requested that the District reduce the optimal head water elevation at S-25B from 2.8 feet NGVD to 1.5 feet NGVD. Since such a change in operational criteria may only provide marginal or negligible flood control benefits at Sweetwater while also resulting in excessive drainage of the surficial aquifer within the eastern C-4 basin, a hydrologic and hydraulic study was conducted to evaluate the response of canal stages and ground water levels to the proposed change in control elevation. This was accomplished through the development and application of an integrated ground water / open-channel flow model spanning the north Miami-Dade county area. The model was based on the MODNET code developed by the District and WEST Consultants (Walton et. al 1999).

The integrated model was used to simulate three operational scenarios for S-25B. The first scenario depicted constant, typical wet season conditions along with the associated equilibrium ground water levels and canal stages. Here, the model simulation commenced with the head water elevation at S-25B fixed at 2.8 feet and was continued until ground water levels and canal stages within the C-4 basin were at or near equilibrium levels. The constant stage of 2.8 feet was subsequently reduced to 1.5 feet and the simulation again proceeded to equilibrium conditions. Ground water levels and canal stage profiles associated with the two control elevations were compared. Stage drawdowns in C-4 were on the order of 0.1 foot in the vicinity of Sweetwater while larger drawdowns on the order of a foot were evident further east near Blue Lagoon. Furthermore, water table drawdowns were fairly widespread around S-25B but were negligible at Sweetwater.



The second and third scenarios covered a historically wet period of record spanning the last three months of 1994. This included a storm event that occurred in mid November of that year. The model simulations for these scenarios were essentially the same except that the target head water elevation for S-25B was set at 2.8 feet in scenario 2 until about the time of the storm event, when it was reduced to 1.5 feet for the remainder of the simulation. In scenario 3, the latter target head water elevation was specified for the entire simulation. A comparison of the simulation results revealed that the difference in control elevation at S-25B resulted in pre-storm stage differences of about 0.1 – 0.2 feet near Sweetwater while stage differences of about 0.5 – 1.0 foot were evident in Blue Lagoon. This is consistent with the results obtained from scenario 1. In addition, differences in water table elevation within the eastern C-4 basin a week prior to the storm were generally about 1 foot near the canal and close to 0.5 foot about one mile away. In contrast, differences in water table elevation within the Sweetwater area were only about 0.1 – 0.2 foot.

Using the model results discussed above, District engineers make the following conclusions and recommendations:

- The proposed change in target head water elevation for S-25B may provide development in the eastern C-4 basin with some extra protection against periodic flooding during the wet season due to smaller but more frequent storm events.

- The change in operational protocol would most likely provide negligible flood control benefits for the town of Sweetwater as well as other municipalities located further west. The simulation of a minor (1 in 3 year) flood event revealed a reduction in peak stage of less than 0.05 foot. For extended operation of S-25B at the lower control elevation under typical wet season conditions, it was found that canal stages within the vicinity of Sweetwater decreased by only about 0.1 foot.

- Under typical wet season conditions, the lower control elevation for S-25B will significantly lower the water table within the eastern C-4 basin, providing approximately an additional 1,800 acre feet of storage. The impacts of the resultant drawdowns on water supplies should be investigated prior to implementation. In particular, it should be verified that no violations of MFL criteria would occur.

- Under typical wet season conditions, approximately 50% of the potential increase in storage will occur within one week after lowering the control elevation at S-25B while approximately 90% can be achieved within a month.

- Under typical wet season conditions, maintaining the lower control elevation over an extended period of time can increase the discharge through S-25B by about 40%. This is a significant increase in basin outflow.

- Issues regarding local hydrologic impacts should be addressed through more detailed modeling. Furthermore, implementation of the modified operation criteria should be accompanied by an appropriate monitoring plan.



- Limited data indicate that a significant constriction in the channel cross section exists in C-4 at a location between the Palmetto Expressway and West Flagler Street. Its length has not been determined. Additional data should be acquired to more accurately determine the nature and extent of this constriction.

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## Introduction and Purpose

Background

The C-4 basin is approximately 61 square miles in area and is located in central Miami-Dade county (**figure 1**). It is drained by not only the C-4 canal but also several secondary canals operated and maintained by Miami-Dade county (**figure 2**). Land uses within the basin include regional wetlands, limestone mining and urban. In particular, a number of municipalities can be found along the C-4 canal reaches located east of the Florida Turnpike. These include Sweetwater, West Miami, Westchester and Town Park. These communities have historically experienced frequent flooding during the wet season. Severe flooding occurred during Hurricane Irene and the unnamed storm of October, 2000.

Stages in the C-4 canal are controlled primarily through structure operations and adjacent ground water levels. Structures influencing flow in C-4 include S-25B, G-93 and, to a lesser degree, S-22 (**figure 2**). Structures S-25A, G-119 and S-121 could also influence stages in the C-4 basin but are seldom opened. S-25B is the primary control structure for the basin. The canal network within the C-4 basin also contains several quarries that provide in-line storage for canal flows. These include Dolphin Lake, Blue Lagoon, Angler Lake and two unnamed borrow pits located at the junction of C-3 and C-4 (figure 2).

Additional information on the hydrologic, hydraulic and operational characteristics of the C-4 basin can be found in USACOE(1952), USACOE(1954) and Cooper et al (1987).

Purpose and Scope



In anticipation of reducing the frequency and severity of flooding within the town of Sweetwater, local officials have requested that the District reduce the optimal head water elevation at S-25B from 2.8 feet NGVD to 1.5 feet NGVD. Such a change, however, in operational criteria might provide only marginal or negligible flood control benefits at Sweetwater while also resulting in excessive drainage of the surficial aquifer within the eastern C-4 basin. To address these issues, a hydrologic and hydraulic study of the C-4 basin needed to be conducted in order to evaluate the response of canal stages and ground water levels to the proposed change in control elevation. In particular, the effort focused essentially on providing preliminary answers to the following questions:

- Will lowering the target head water elevation at S-25B from 2.8 feet to 1.5 feet NGVD under wet hydrologic conditions result in significantly lower water levels near Sweetwater?
- How far upstream from S-25B will canal stages be influenced by the proposed change?
- To what extent will water table drawdowns occur within the eastern C-4 basin?

These issues are not new and similar solutions have been proposed in the past. In fact, previous analyses of the proposal (Khanal, SFWMD, 1983; Burns & McDonnel, 1991) concluded that little benefit would be obtained by lowering S25b upstream stages beyond current operational

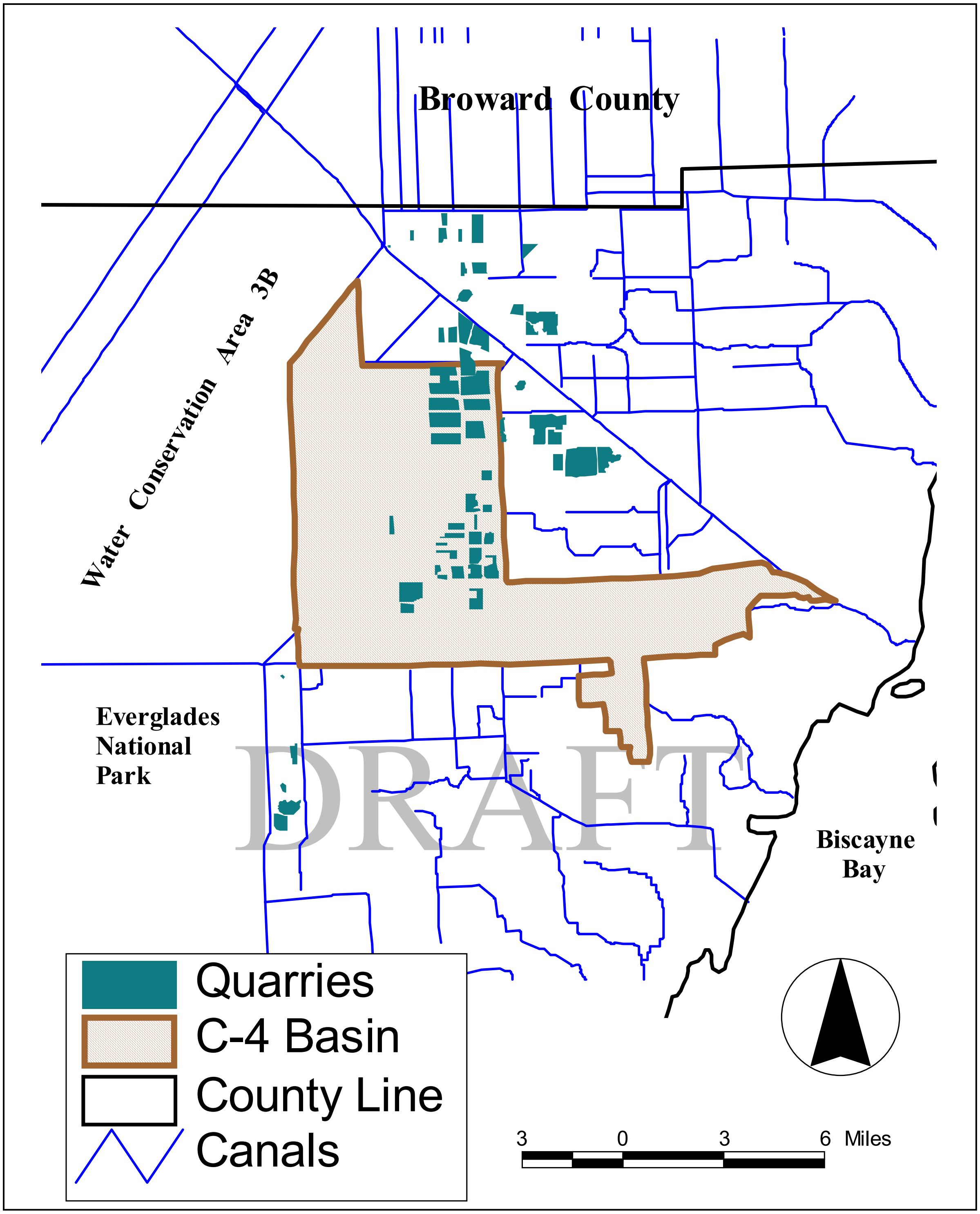


Figure 1. Location of the C-4 Basin

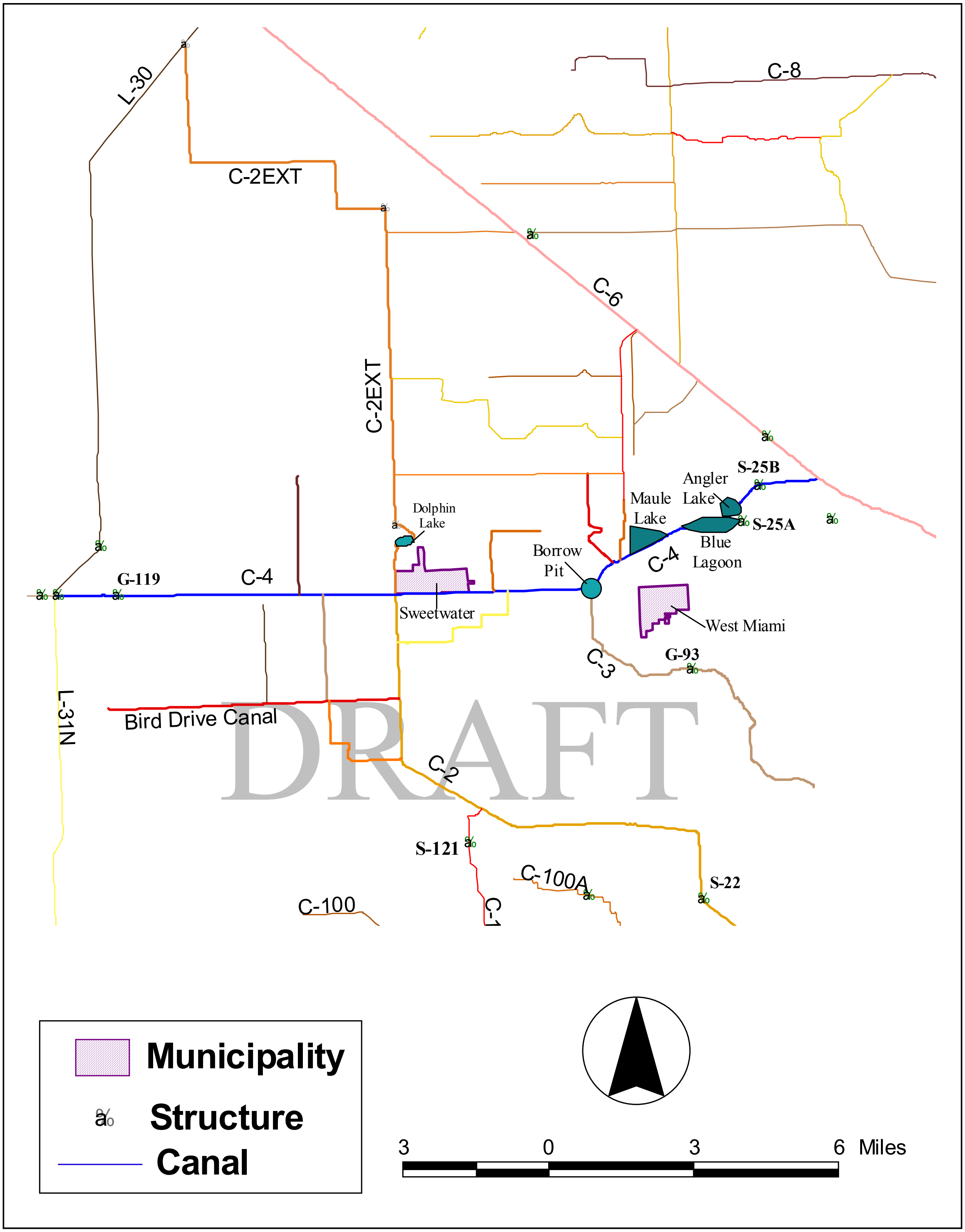


Figure 2. Canals and Structures

settings. In fact, Khanal(1983) concluded that benefits would be limited to a 0.1 foot lowering of peak flooding depths. Burns & McDonnel(1991) confirmed Khanal's findings. However, these earlier analyses were based on models that are simpler than current ones while certain hydraulic information incorporated into these earlier studies was of lesser quality than more recent data. Given the nature of the issues along with the strong hydraulic interactions that exist between the canals and the surficial aquifer, it was determined that a new integrated ground water / open-channel flow model would be the most appropriate tool for evaluating the benefits and impacts of reducing the optimal wet season control elevation for S-25B. A detailed description of both the model and the applications pertinent to this study are provided in the sections that follow.

**Model Development**

The development of a linked ground water / open-channel flow model generally requires three phases. These are:

(1) Preliminary development of the ground water flow model;

(2) Preliminary development of the open-channel flow model;

(3) Development of the integrated model through linkage and subsequent refinement of the ground water and open-channel flow models.

<u>Ground Water Flow Model Development</u>

The ground water flow model employed in this effort is a modification of the north Miami-Dade county MODFLOW model developed previously by District staff (**Wilsnack et al 2000**). A number of significant modifications were made to minimize simulation times and reduce the overall computational burden of the model. These included changes to the outer boundary, the model grid and the river package.



*Outer Model Boundary*

The active model domain was reduced in area from the one specified in Wilsnack et al (2000). The northern boundary was relocated from the C-11 canal to the C-9 and C-304 canals. The rest of the outer boundary was essentially unmodified.

*Model Grid*

The model grid was rediscretized horizontally from 500 x 500 feet to 1000 x 1000 feet. This greatly reduced the number of active cells. The grid's orientation and location were maintained. No changes to the vertical discretization were made.

*MODFLOW Packages*

The canal reaches that were included in the open-channel routing model were removed from the River package. Input data sets for the Well, Lake, Drain and Wetland packages were merely reconstructed to reflect the revised model grid. Similarly, the arrays pertaining to aquifer properties and wetland parameters were reconstructed for the revised grid by averaging the four cell values of the previous grid within each cell of the current grid.

*History Matching*

The revised ground water flow model was used to simulate the same period of record that served as the basis for calibration of the previous model (July 4, 1993 – December 31, 1994). This was done to help verify that the ground water portion of the integrated model was adequately constructed. Wetland stages and ground water levels computed by the model were compared to measured water levels at selected monitoring sites. This evaluation was only carried out in a qualitative fashion to help detect any errors in the revised ground water model.

*Sensitivity Analysis*

A sensitivity analysis of the previous version of the model is provided by **Wilsnack et al (2000)**. No additional sensitivity analyses were performed on this version of the ground water model.

Open-Channel Flow Model Development

An open-channel flow model of selected canal reaches in the C-2, C-3 and C-4 basins was constructed using the UNET code. This model expands the one previously developed by **Wilsnack (2001)** for the C-2 Extension canal. It essentially includes all reaches of the C-4 canal located between structures G-119 and S-25B; the portion of C-3 upstream of G-93; the C-2 canal between C-4 and C-100C; and various secondary canals such as the C-2 Extension canal and the Bird Drive canal (figure 2). A reach-node diagram depicting the conceptual model is shown in figure 3.



*External Boundaries*

Flows through G-119 and DERM-L30, the structures sited at the upstream ends of C-4 and C-2 Extension, respectively, were used to establish flow boundary conditions at these upstream boundaries. Zero flow boundaries were specified at the upstream ends of all terminal reaches. Similarly, staff gage readings taken at the confluence of C-2 and C-100C were used to establish a stage boundary condition at the downstream end of C-2. At the downstream ends of C-3 and C-4, stage boundaries depicting the G-93 and S-25B tailwater hydrographs were applied. Spillways were sited at a short distance upstream of these boundaries where gate openings computed from measured head water, tail water and flow data were specified so as to enable the model to approximate both measured head water stages and flows at these downstream boundaries. This type of downstream boundary was only used at S-25B for calibration purposes. In applications of the model, S-25B was operated in a manner that resembled standard operational protocol.

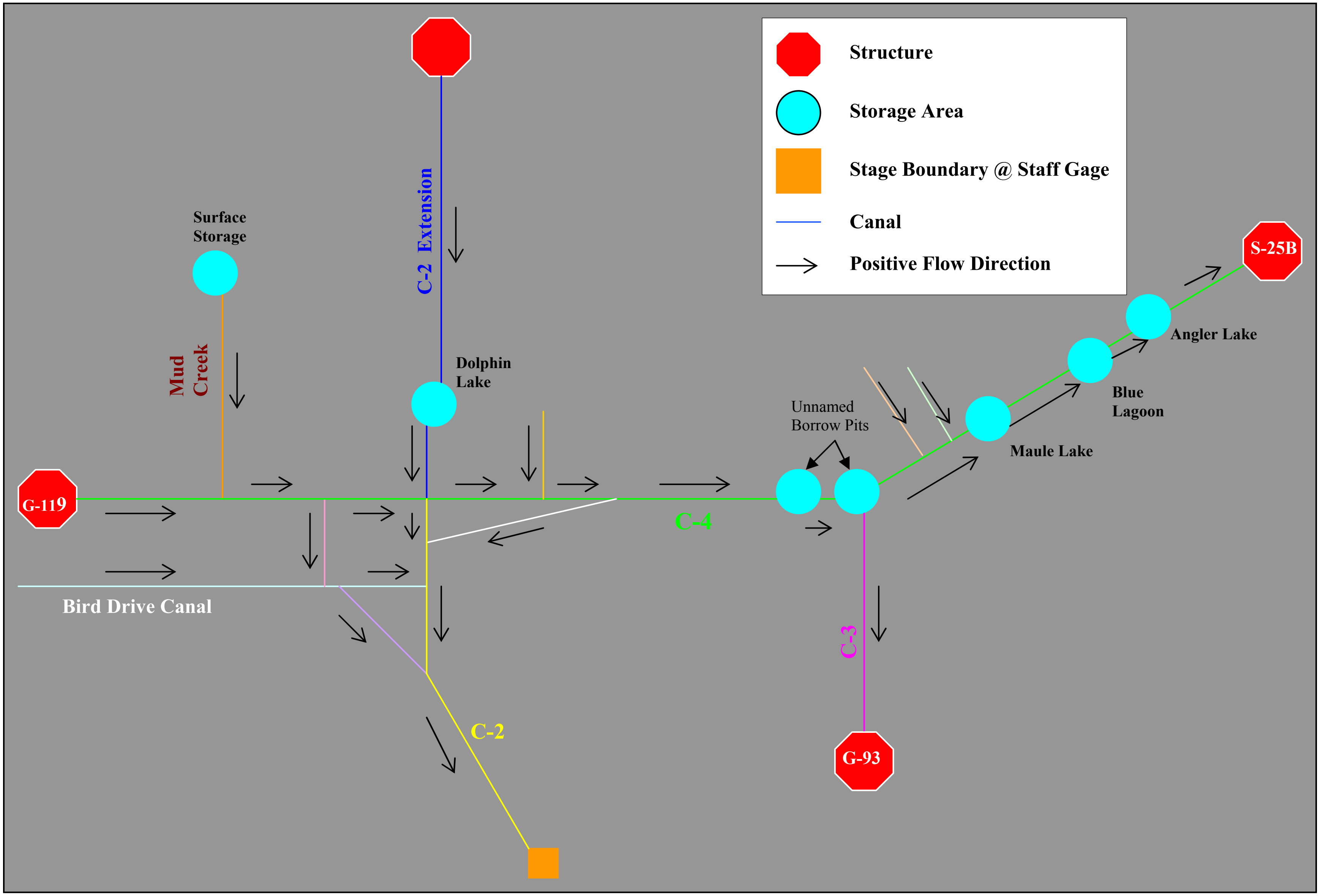


Figure 3. Reach – Node Diagram of the UNET Model

*In-Line Channel Storage*

The borrow pits excavated into the C-4 canal corridor (**figure 2**) were conceptualized as in-line storage areas (**figure 3**). The Dolphin Lake excavated along the C-2 Extension canal was treated in like manner.

*Ground Water Interactions*

The UNET code allows for interactions between a ground water boundary and canal flow through use of an explicit formulation (HEC, 1997). In the preliminary UNET model, the ground water boundary was based on measured ground water levels assigned to each canal reach. Base flows were simulated using the reach transmissivity concepts formulated by Chin(1990) for canals in Miami-Dade county. The results of this study were used to compute reach transmissivity values for each canal reach as outlined by **Wilsnack(2001).**

*Geometric Data*

Canal cross sections were constructed using survey data obtained by SFWMD, the USACOE and Miami-Dade county DERM. The locations of measured cross sections are shown in **figure 4**. Most of these cross sections did not extend beyond top-of-bank. Also, at the in-line storage areas, measured bottom elevation data were obtained but were sparse and inaccurate due to the interference of hydrilla on the soundings.

A GIS coverage of the canal centerlines was digitized in ARC/INFO using the USGS DOQQ images with 1 meter resolution. Surveyed canal centerline locations were available for a few reaches.



*Channel Roughness*

The determination of appropriate values for Manning's n was somewhat difficult due to limited data as well as the presence of vegetation that clog the channels to varying degrees. The capacities of the canals within the study area can (and often do) become substantially reduced by the growth of hydrilla. Growth heights of about 60% of the water depth are typical (personal communication, Miami field station staff). Furthermore, hydrilla are mechanically or chemically removed from the primary canals within the study area several times per year. It follows that the actual degree of clogging with vary throughout the year not only due to maintenance cycles but also variation in factors that influence growth, such as water depths and daylight hours. Under optimal conditions, growth rates of about 1 inch per day are possible (Gordon Baker, Vegetation Management Division, personal communication).

All of this suggests that the effective Manning's n values for the modeled canal reaches will vary significantly throughout the year. Values of n for SFWMD project canals have been published by the USACOE (1953) and Stephens et al (1962). For canals constructed in limestone, n values were reported to vary from about 0.02 for reaches with little vegetative growth to nearly 1.2 for reaches with heavy vegetation. As expected, the higher values occurred in canals with smaller wetted areas. While compiling published ranges of n for various channels throughout the world,

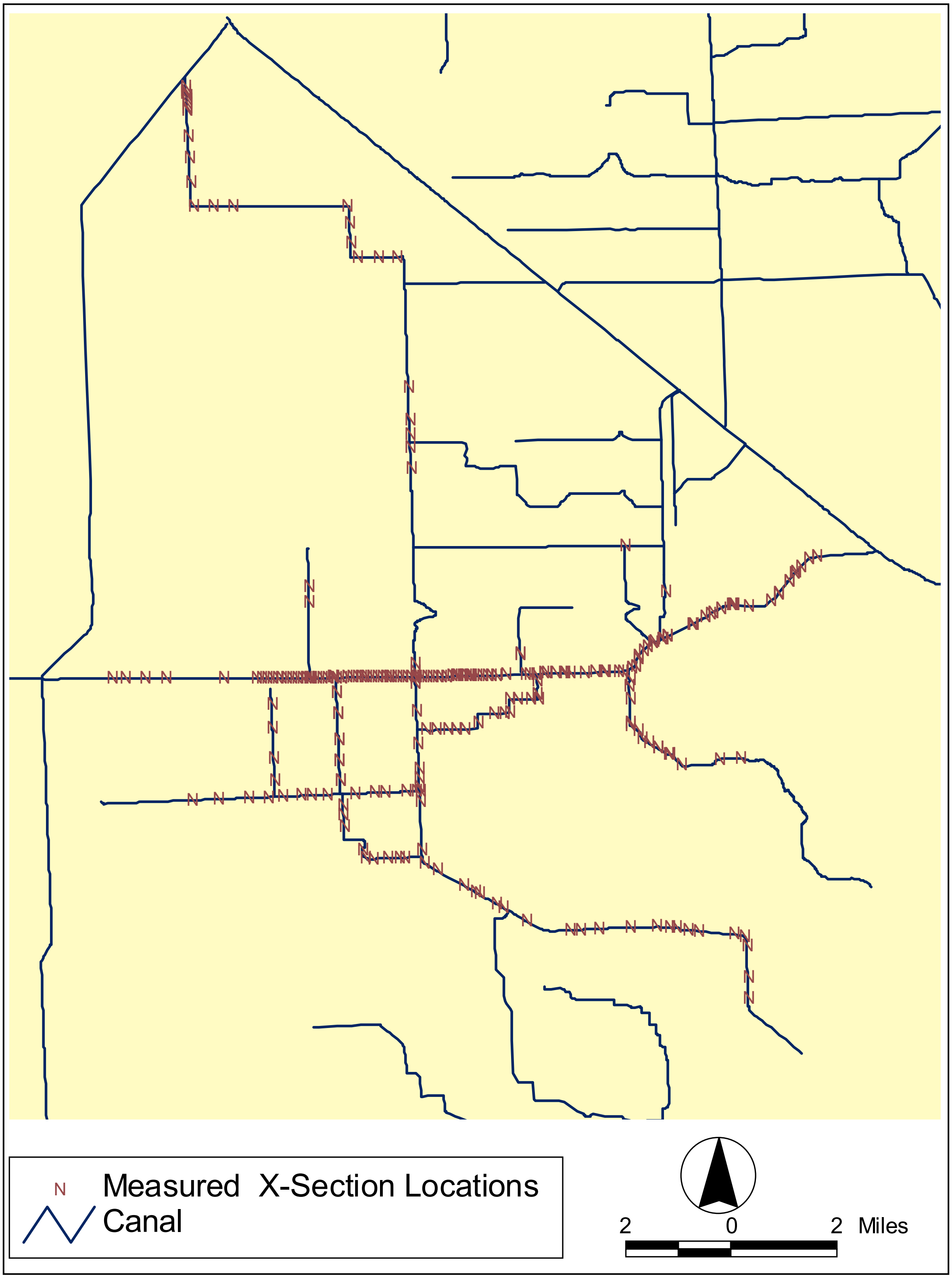


Figure 4. Locations of Measured Canal Cross Sections

Stephens et al (1962) classified channels into three categories: large (hydraulic radius > 20 feet), moderately sized (~200 – 2,000 sq. ft. wetted area), and small (< 200 sq. ft. wetted area). While the variation in n for large channels turned out to be minor, significant variations in n were found to occur in moderate and small channels due to varying degrees of vegetative growth. Furthermore, Stephens et al (1962) verified through field experiments conducted on C-4 that Manning's n varies with the product of mean velocity and hydraulic radius. This is consistent with previous findings published by Ree and Palmer (1949) and Ree (1957) for flow in vegetated waterways. Consequently, it appears that the Manning's n range of 0.03 – 0.035 traditionally used in design and analysis of SFWMD canals is not appropriate for the purposes of the current modeling effort. In fact, the findings and recommendations published in USACOE (1953) seem to suggest that values of n in this range are generally representative of channels constructed in sands and nearly free of vegetation. Higher values would be more reflective of conditions found in the C-4 basin. Therefore, in the initial development of the UNET model, Manning's n ranged from 0.01 – 0.02 for the very deep canal reaches of C-2 Extension to 0.04 – 1.0 elsewhere.

Integrated Model Development

With preliminary versions of both the MODFLOW and UNET models constructed, only one additional data set was needed to link the two models via the MODNET code. Included in this data set were parameters that depicted the degree of interaction between the various canal reaches and ground water. Also included in the data set were the geometric specifications needed to enable the UNET storage areas to also function as Lakes in MODFLOW.

*Ground Water / Surface Water Interaction Parameters*



In MODNET, the degree of hydraulic interaction between channels and ground water is represented by the HCOND parameter. The values of HCOND input to the model reflected estimated reach transmissivities and were computed in ARC/INFO using aquifer transmissivities along with constructed channel cross sections (see, for example, **Wilsnack, 2001**). The UNET storage areas that were linked to ground water through MODFLOW lakes were limited to Angler Lake, Blue Lagoon and Maule Lake. The required MODNET input data for these storage areas are essentially the same as the input required for the standard MODFLOW lake package. These include values of conductance that depicts lake / aquifer interaction, addresses of MODFLOW grid cells that adjoin the lakes, and extraneous inflows/outflows such as rainfall, ET and runoff.

*Model Discretization*

As indicated previously, the cell size of the ground water model grid was set at 1000 feet by 1000 feet to keep the computational burden of the model manageable. Some of the numerical difficulties cited by **Wilsnack et al (2000)** in development of the ground water flow model were partially due to the sheer number of active cells in the model. It was initially anticipated that if a UNET model of the C-4 basin were linked to such a ground water model, run times would be prohibitively long and numerical problems could abound. As a result, it was felt that some compromise in model resolution was needed, albeit at the expense of computational accuracy. **Appendix D** contains a detailed analysis of the types of errors introduced by the discretization. In particular, a MODFLOW time step of 1 hour was used consistently throughout the study while the UNET time step lengths were typically 10 or 15 minutes. Although the latter exceeds the courant condition (**Appendix D**), the model did not appear to be sensitive to the time step length (**Appendix E**).

Calibration

Calibration of the MODNET model involved the history matching of ground water levels, canal stages and canal flows over the period of record beginning July 4, 1993 and ending December 30, 1994. This is the same period of record used to calibrate the northern Miami-Dade county ground water flow model. It was used in this effort as well in order to help minimize the time and effort required for history matching. While this period of record does not contain any hurricanes or major storms such as Irene or Storm 2000, it does contain a storm that was large enough to raise concerns over flooding within the town of Sweetwater.

Agreement between measured and computed stages and flows was improved by adjusting reach transmissivity, rainfall recharge and canal conveyance. As discussed previously, canal conveyance can vary significantly throughout the year due to periodic growth and harvesting of hydrilla. Furthermore, local changes in conveyance can result from debris clogging bridges during storm events as well as submerged automobiles in reaches with smaller cross sections. Unfortunately, no records that would substantiate any specific incidences of these latter effects were located. In contrast, canal maintenance records were available from the SFWMD Miami field station to document time frames when canals underwent hydrilla removal, although such records did not indicate the specific canal reaches that were serviced on a given date. Similar data were supplied by Miami-Dade county for the Bird Drive, C-3, and C-2 Extension canals. Considering the general practice of beginning maintenance operations at the upstream end, the service records were used to construct a time series of conveyance adjustment factors that could be applied and modified as needed during calibration. The UNET code was modified so these factors could be specified as an arbitrary time series as opposed to on a seasonal basis. This was necessary since canals are not necessarily maintained the same way each year.



**Figure 5** shows the canal stage and flow monitoring sites that are not located at a structure. Also shown are the monitoring stations for ground water levels and wetland stages. **Appendix A** contains the computed versus measured canal stage hydrographs for each monitoring site. Appendices **B** and **C** contain the canal flow and ground water level hydrographs, respectively. Computed canal stages and flows agree fairly well with the observed values while also matching the trends. However, it is evident that some of the hydrograph peaks are missed or poorly replicated by the model while rapid oscillations in stage or discharge are smoothed over. This may be partially due to the fact that historical S-25B and G-93 operations were not precisely represented in the model. At either structure, the time series of gate openings entered into UNET only included positive values reflecting times when the structures discharged to tide under a head differential greater than 0.1 foot. The input data were restricted in this way in order to help prevent flow reversals and oscillations that could lead to instabilities. Furthermore, measured data indicating large flow reversals are suspect. It is also of interest to note that some of the inherent discrepancies between measured and computed structure flows apparently resulted in the large differences between computed and observed head water stages that occurred at S-25B during August and September of 1994. During this time frame, the gate was closed in the model while measured data depicted an open gate with oscillating flow. Similar effects can be seen at G-93.

In general, the use of an implicit weighting factor ($\theta$) equal to one in UNET can attenuate peaks in the computed hydrographs. This has been demonstrated in theory by Fread (1974) and in practice by Barkau (2001). In fact, Barkau (2001) indicates that a value of 0.6 should be used in practice where possible in order to enhance accuracy while also avoiding the types of numerical oscillations that can occur when $0.5 \leq \theta < 0.6$. Nonetheless, a value of one for $\theta$ was used consistently in this study since lesser values lead to instabilities in the model. This may have made some small contribution to the attenuation of computed stage peaks.

A closer inspection of the computed and measured stage hydrographs (**Appendix A**) at S-25B reveals a slight bias where computed head water elevations are often slightly lower than measured. A similar observation appears to be true of the discharges. In contrast, computed stages and discharges at G-93 tend to be higher than measured data. While this would seem to suggest that the modeled conveyance of C-3 is too high, conveyance values for this channel were decreased during the calibration process within realistic limits. Other explanations include systematic errors in spatial rainfall and aquifer parameters. Similar biases are also evident in computed stages at the C-4.CORAL gage. In this case, however, the bias seems to go from low to high during certain time frames. Likewise, computed stages at T5 tend to be higher than measured until about late May, 1994. Afterward, they are biased on the low side. It will probably be difficult to resolve these types of systematic biases without improving the accuracy of rainfall recharge and increasing model resolution.

The computed ground water level hydrographs (**Appendix C**) appear to follow measured trends fairly well and, during much of the simulation, are within the 0.5 foot tolerance set previously for the ground water model (**Wilsnack et al, 2000**). However, some systematic biases are evident here as well. In fact, a comparison of the residual hydrographs for NESRS3_B, G-3439, G-3572 and G-3570 with G-3568, G-3328 and G-3329 indicates that computed ground water levels are almost consistently low just south of C-4 and are biased on the high side just north of C-4. Within the C-2 basin, this may be at least partially due to errors in the computed canal stages. Unfortunately, there are no data to verify this. Errors in rainfall recharge, aquifer parameters and canal stages applied to the River package could also be contributing factors.



Sensitivity Analysis

A sensitivity analysis was conducted on the model in order to assess the impact of changes in selected input on model output. For brevity, only C-4 stages and flows were addressed since these were the system responses of primary concern in this study. Likewise, the input data varied were limited to $\Delta t$, $\Delta x$, hydraulic conductivity, reach transmissivity and rainfall recharge. Furthermore, parameters were varied independently from each other in order to acquire a feel for how uncertainty in each parameter can translate to uncertainty in model output. Correlation between the various model input data was not considered here as it should be in a more rigorous sensitivity analysis.

Table 1 provides a quantitative summary of the sensitivity analysis results. **Appendix E** contains

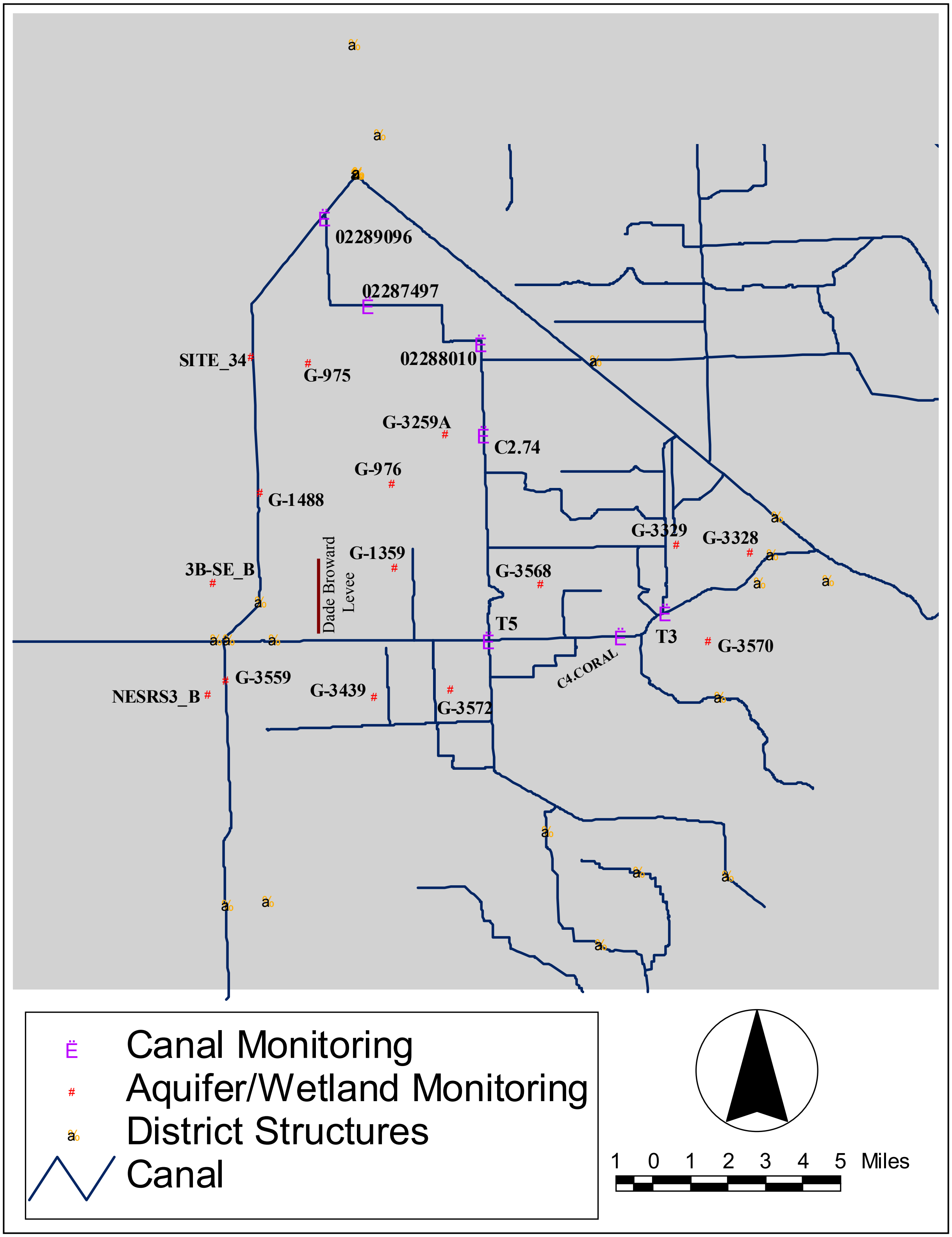


Figure 5. Water Level and Flow Monitoring Sites

the stage and flow hydrographs that quantitatively depict model sensitivity.

Table 1. Sensitivity of Stages and Discharges in C-4 to Changes in Selected Parameters

| Parameter | Comments on Model Sensitivity |
|---|---|
| UNET $\Delta t$ | Both stages and discharges were insensitive to $\Delta t$ between 5 and 15 min. |
| UNET $\Delta x_{max}$ | Stages and flows increased when a $\Delta x_{max}$ = 500' was specified. Sensitivity was greater at the D-B levee junction than at T5 and other locations further east, where the model was not very sensitive to the specified $\Delta x_{max}$. This is at least partially due to the larger number of interpolated nodes added to reach # 1 (8) as opposed to the other reaches (2 or less). |
| Aquifer K | Both stages and flows were sensitive to K. The sensitivity was greater along the more western reaches. Also, the model was less sensitive to increases in K than it was to decreases in K. |
| Reach Trans. | Sensitivity was similar to that for K, but lesser in magnitude. |
| Rainfall Recharge | This appeared to be the parameter the model was most sensitive to. Under higher flows, stage sensitivity increases somewhat to the west while discharge sensitivity increases somewhat to the east. |

## Model Applications

The Executive Office of the SFWMD directed staff to evaluate the potential flood control benefits and water supply implications of lowering the secondary wet season control elevation of S-25B by 1.3 feet. The intended purpose of the modified operation was to reduce flood stages within the C-4 basin. The flood control benefits were investigated by performing and analyzing three model based scenarios. The first scenario compared the canal stages and ground water levels that would result by establishing either control elevation at S-25B under equilibrium wet season conditions. The second scenario depicted the operation of S-25B at the higher control elevation of 2.8 feet NGVD during the period of record spanning September 27, 1994 to November 14, 1994. The control elevation was subsequently reduced to the secondary elevation of 1.5 feet after the latter date and the simulation was continued to December 30, 1994. Finally, in a third scenario the lower control elevation was maintained throughout the entire period of record. During the month of November, 1994 a minor flood event (with a return frequency of about 1 in 3 years) occurred with the C-4 basin.

<u>Operational Scenario # 1: Changes in Wet Season Equilibrium Conditions</u>

In this scenario, hydrologic stresses that are representative of the wet season were applied to the model and held constant throughout the simulation. The only temporal change was a decrease in target head water elevation at S-25B from 2.8 feet to 1.5 feet NGVD. This change occurred after the model reached near equilibrium conditions with the higher control elevation. The resulting water surface profile in C-4 before the drop in control elevation is shown in figures 6 along with the water surface profile reflecting the lower control elevation. Figures 6a and 6c show the water surface profiles in comparison to the channel invert and bank elevations while figures **6b** and **6d** only include the water surface profiles for clarity. **Figure 7** shows the resultant drawdown in water table elevation. It is evident that most of the drawdown occurs within the eastern C-4 basin, providing a storage increase of approximately1,800 acre feet (this is based on a drainable porosity of 0.15). Drawdowns appear to become less than 0.2 feet upstream of the C-3 junction. In particular, the drawdown near the downstream end of Sweetwater (approx. river mile 8.1) is about 0.1 foot (figure 6b). This may be partially due to a shallow cross section that occurs in C-4 between West Flagler Street and the Palmetto Expressway. The spatial extent of this restriction is not clear and, given the amount of head loss occurring within this reach under hydrologic conditions of interest, it is recommended that additional cross section and profile data be obtained in this area. **Appendix F** provides an additional analysis of the head losses incurred within the subject reach. Included in this analysis are all three scenarios repeated with the shallow cross section removed.

It is also of interest to determine the rate at which the increase in ground water storage is achieved after the S-25B control elevation is reduced. **Figure 8** illustrates the transient nature of the storage increase. It can be seen that about half of the total storage increase is realized one week after the control elevation is reduced while approximately 90% of the total increase is achieved after a month. This information can be useful in determining an acceptable balance between enhancing flood protection and conserving ground water supplies.

The lower control elevation increases the discharge through S25b from 300 cfs to 420 cfs (i.e. a 40% increase). This amount of flow could be regionally significant if continued over an extended period of time. For example, over 6 months the additional 120 cfs equates to more than 40,000 acre-feet of additional drainage.

Operational Scenario # 2: Historical Baseline Conditions

In the scenario used to establish baseline conditions, the simulation commenced at the date indicated previously and continued for 49 days, bringing the simulation to an approximate historical time when the control elevation of S-25B was apparently lowered. After stopping the simulation, it was restarted with the lower control elevation in effect and continued to the final date indicated.

Operational Scenario # 3: Proposed Alternative Conditions

This hypothetical scenario is identical to scenario # 2 except that the lower target head water elevation at S-25B was in effect throughout the simulation. It was also assumed that G-93 was operated in the same manner it was in the baseline scenario.

*Comparison of Scenarios 2 and 3*

Figures **9** through **11** compare the stage hydrographs computed at several locations along C-4. Again, it is evident that any flood control benefits resulting from lower head water stages at S-25B are minimal outside of the eastern C-4 basin. In particular, figures 9 and **10** indicate that the reduction in peak flood stage near Sweetwater is only about 0.02 foot for this event. Similar conclusions can be made regarding increases in ground water storage. Moreover, the lower ground water levels near the eastern end of C-4 may adversely impact public water supplies by increasing the potential for intrusion of saline water. For this reason, a separate analysis was performed by **Ross and Giddings (2003)** to evaluate these types of impacts with respect to the District's Minimum Flows and Levels criteria.



**Summary and Conclusions**

In order to evaluate the potential flood control benefits of a modified operational protocol for structure S-25B, an integrated ground water / open-channel flow model of the north Miami-Dade county area was developed using the MODNET code. The model was used to simulate three operational scenarios. The first scenario evaluated changes in canal stages and ground water levels under typical wet season conditions due to the proposed lower control elevation for S-25B. Near-equilibrium conditions were achieved with the model at each control elevation and the results compared. The second and third scenarios spanned a three-month historical period of record and were essentially the same except that in scenario 2 the target head water elevation at S-25B was set 1.3 feet higher for the first 49 days of the simulation. The results of all three scenarios revealed that most of the potential flood control benefits would be realized within the eastern C-4 basin. West of the C-3 junction, in the portion of the C-4 basin previously named "Area B", drawdowns in canal stages and ground water levels would be minimal ( $\leq$ 0.2 ft ). Consequently, the proposed modifications to the S-25B operational criteria may provide extra flood protection benefits for the more eastern municipalities such as West Miami. In contrast, the more western communities would sustain only minor benefits. Furthermore, the lower ground water levels may pose adverse water supply implications, and a separate analysis should be carried out to address this issue.

**Disclaimer**

The integrated model used to support the above analysis was developed solely for the purpose of addressing the key issues that incited this study. The model may not be suitable in its current form for evaluating the impacts of local scale projects or providing answers to questions that necessitate a high degree of precision. In these cases, additional error and/or sensitivity analyses may be useful in determining whether the model should be used for a given application.

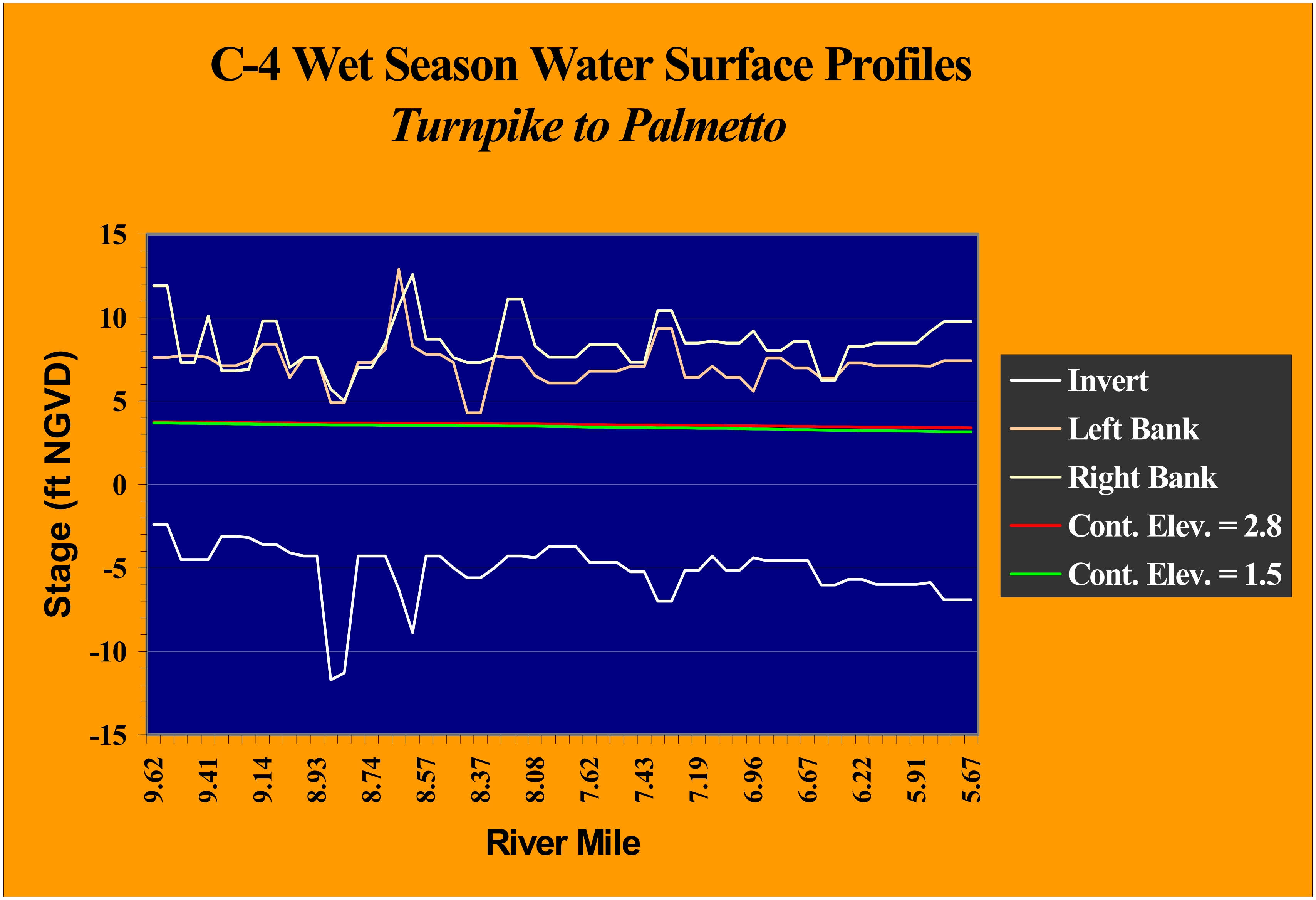


Figure 6a. EquilibriumWater Surface Profiles Under Typical Wet Season Conditions

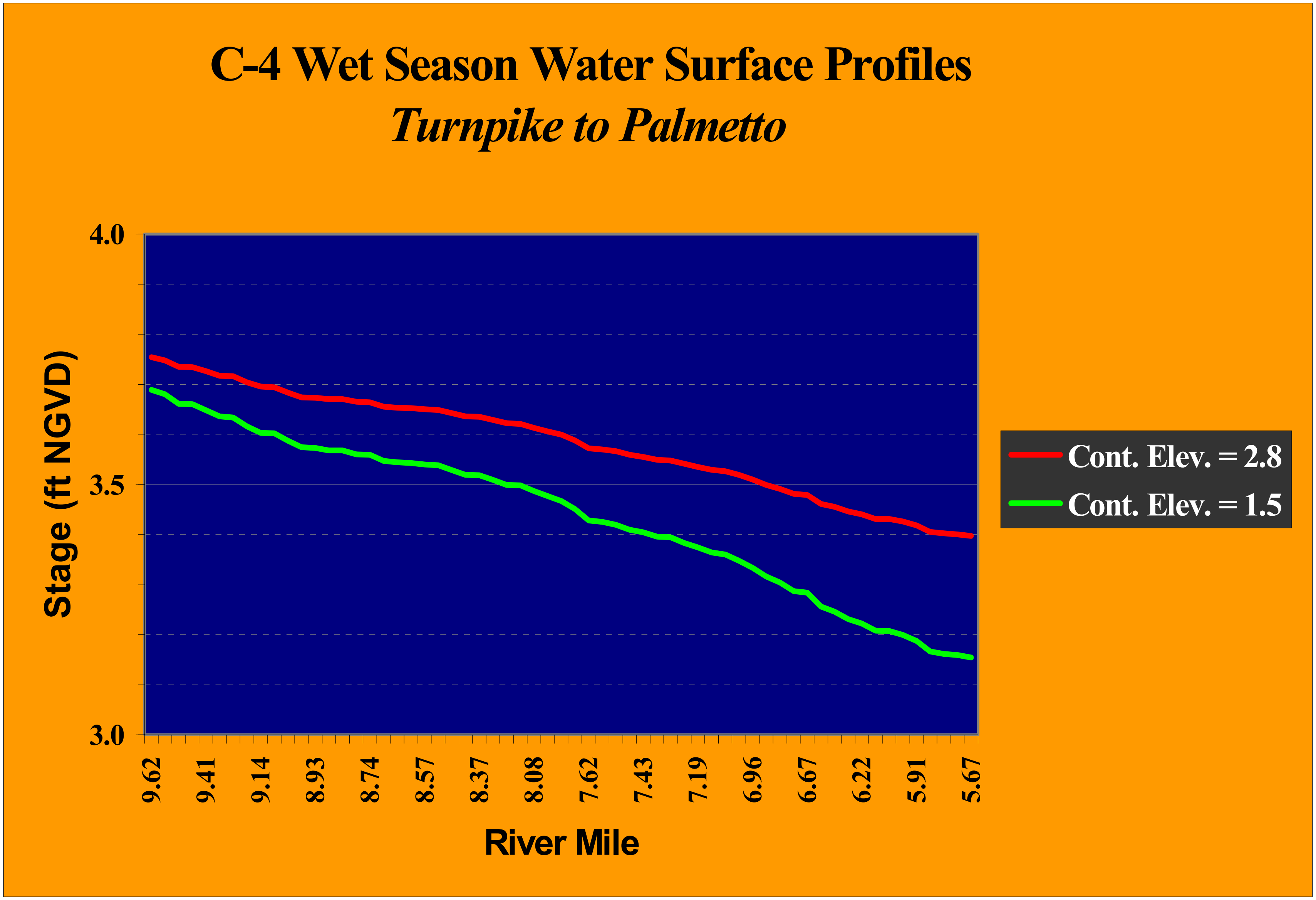


Figure 6b. EquilibriumWater Surface Profiles Under Typical Wet Season Conditions

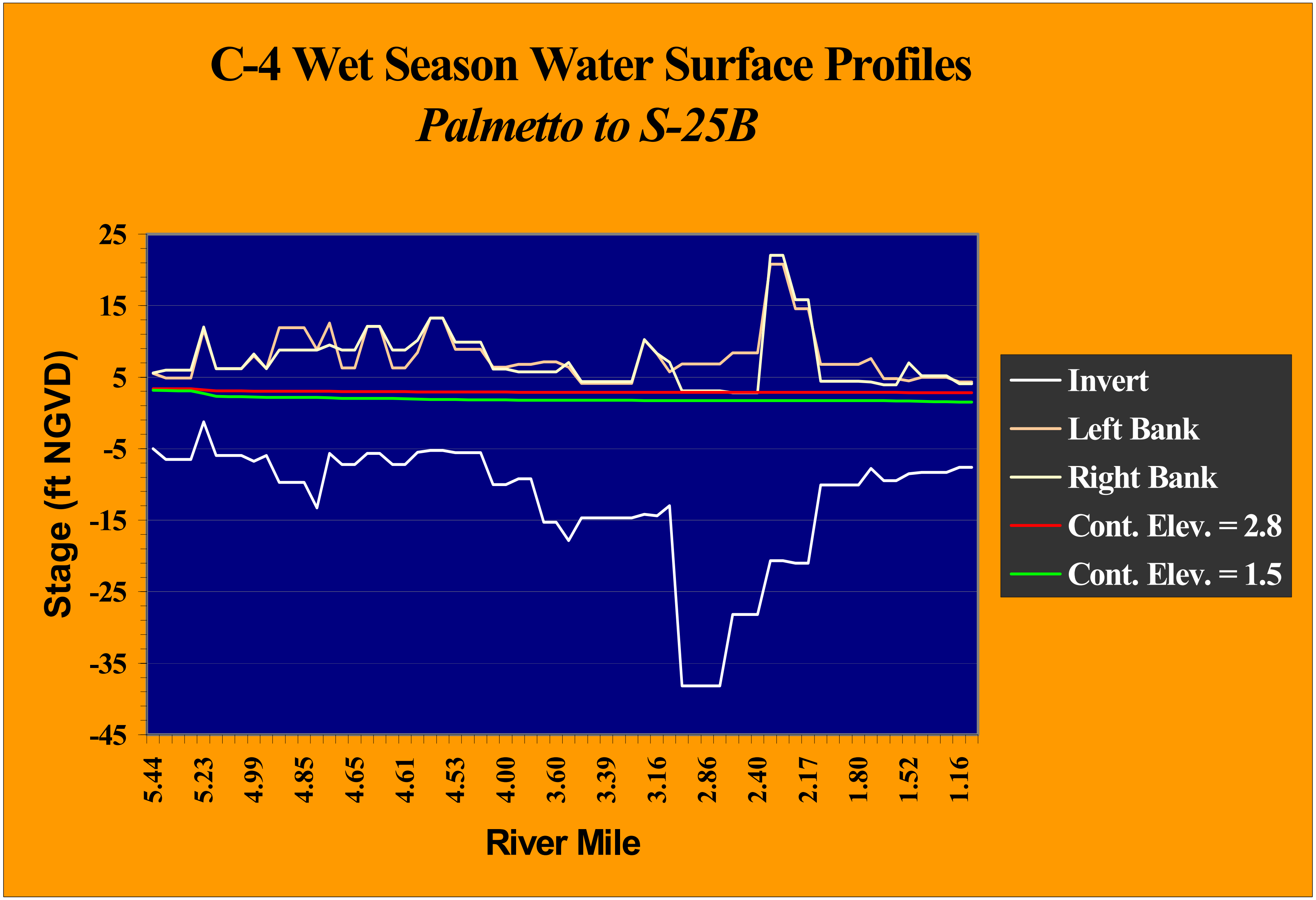


Figure 6c. EquilibriumWater Surface Profiles Under Typical Wet Season Conditions (con't)

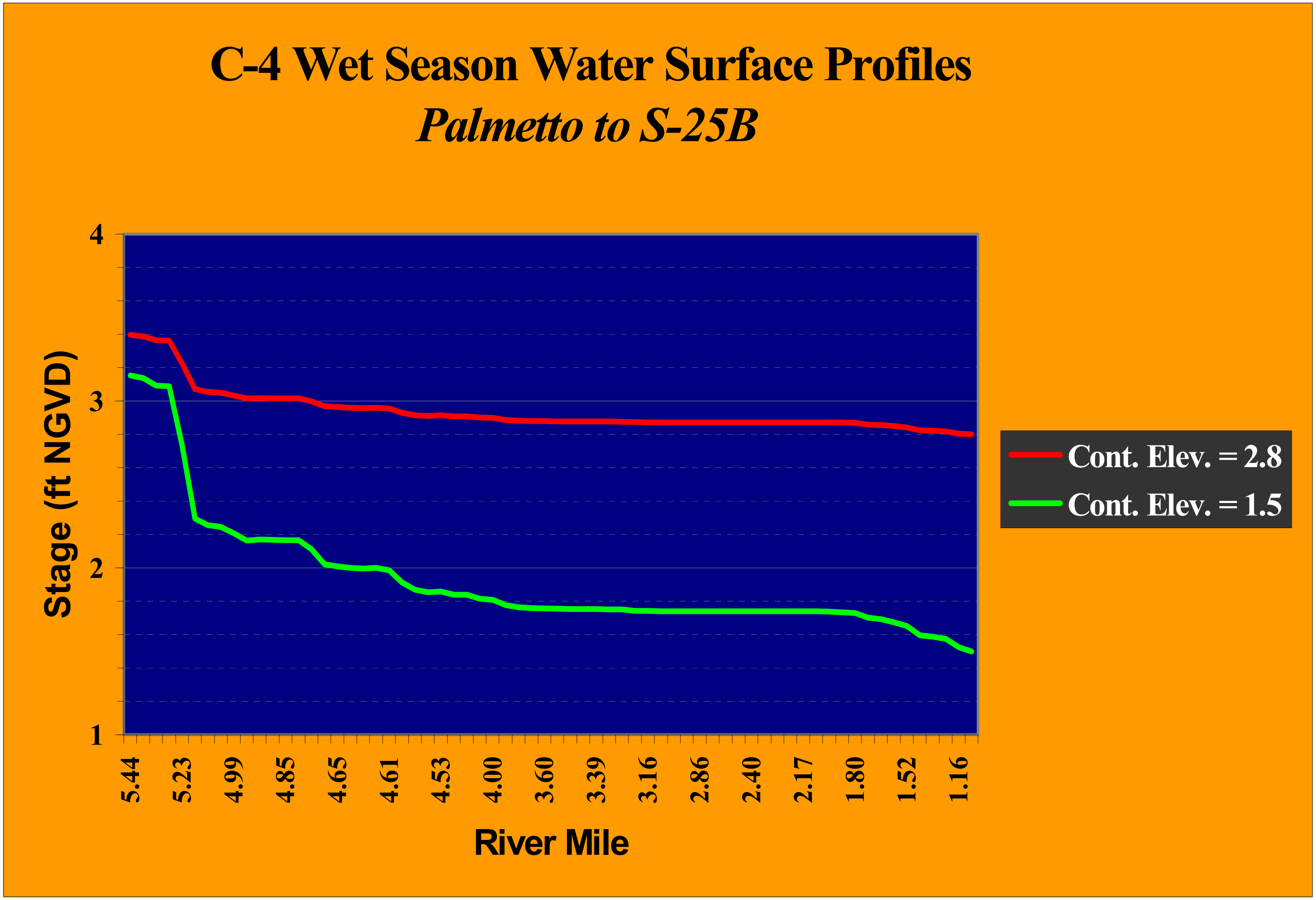


Figure 6d. EquilibriumWater Surface Profiles Under Typical Wet Season Conditions (con’t)

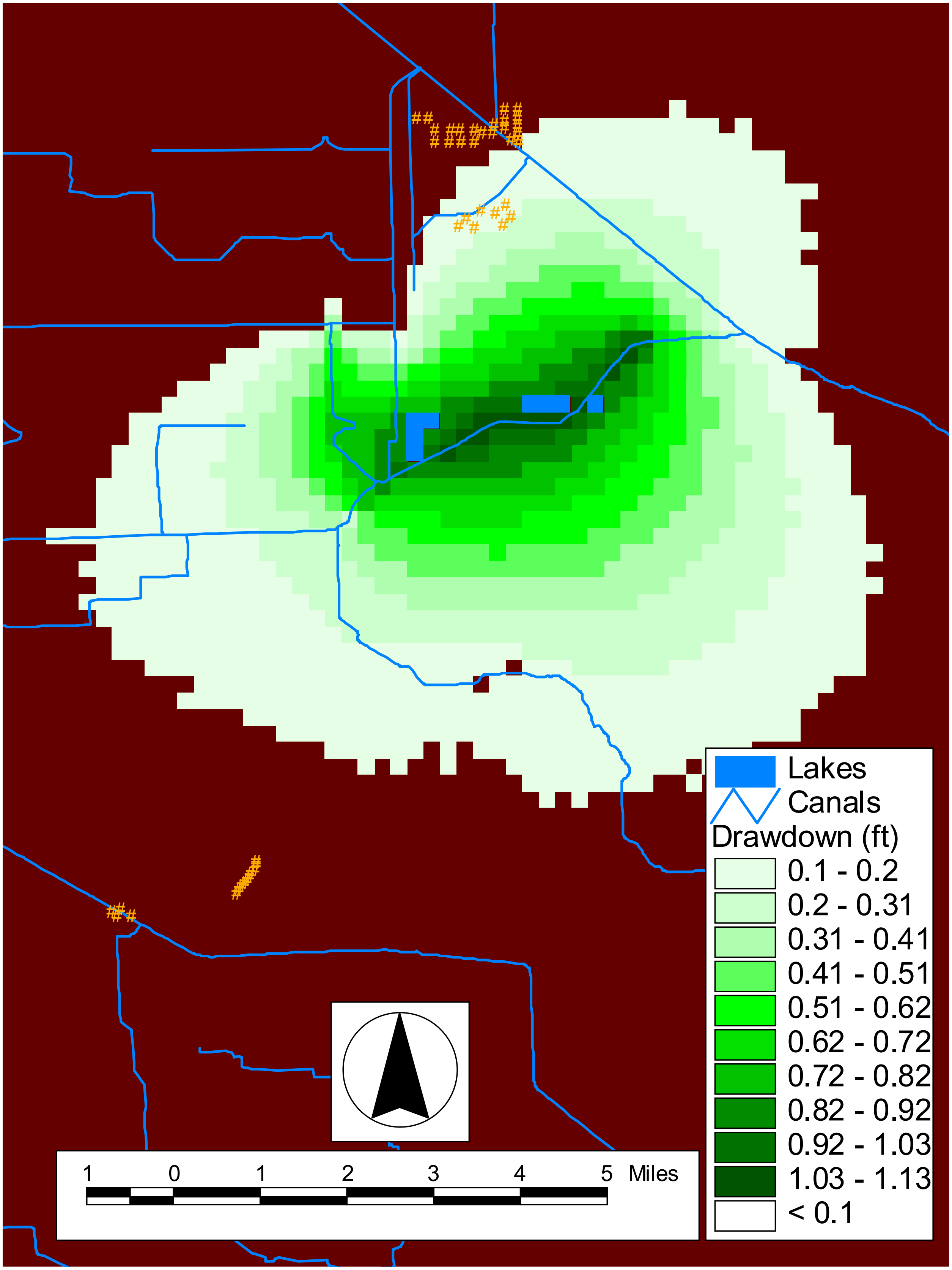


Figure 7. Water table Drawdown Under Equilibrium Wet Season Conditions

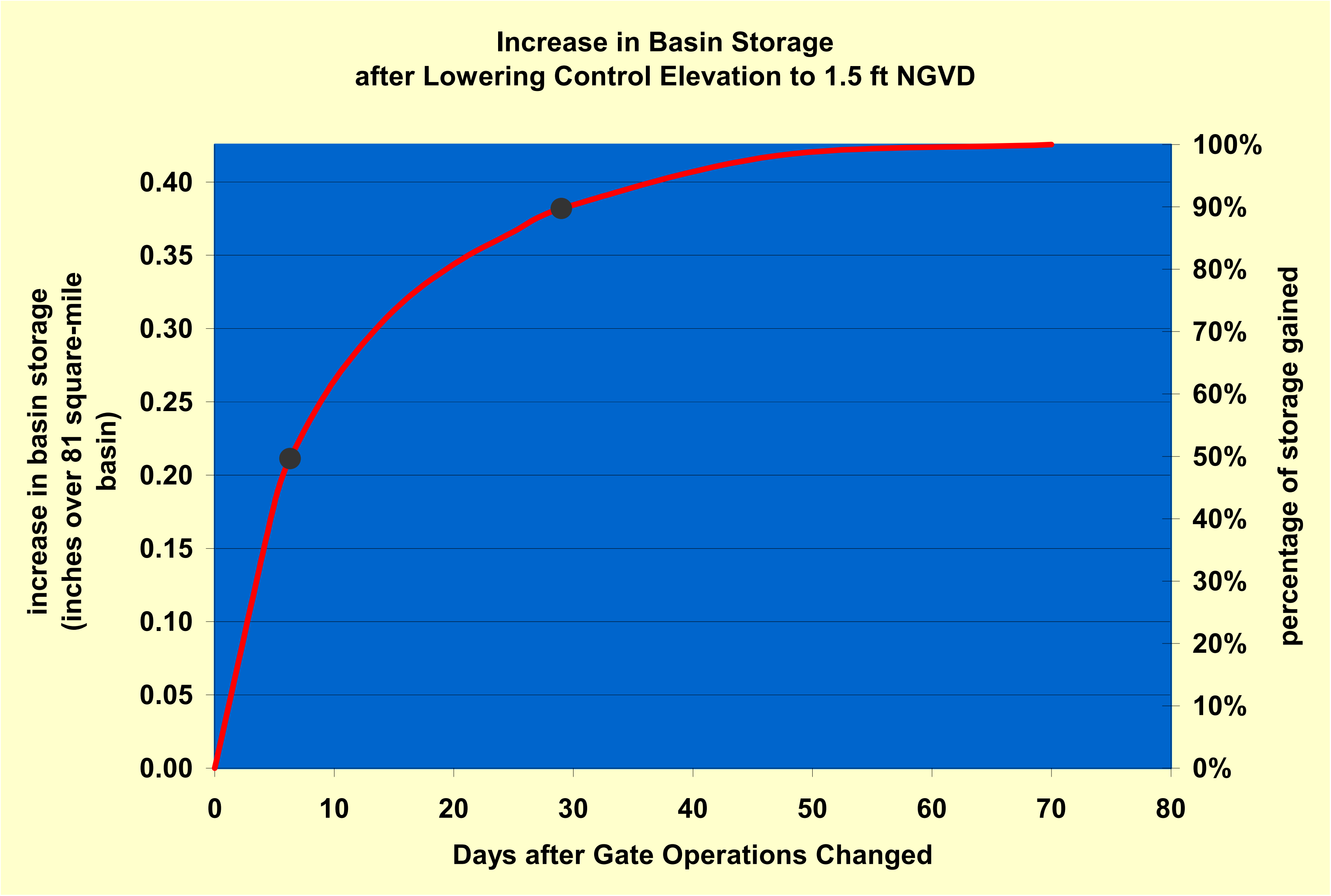


Figure 8. Increase in Storage Over Time

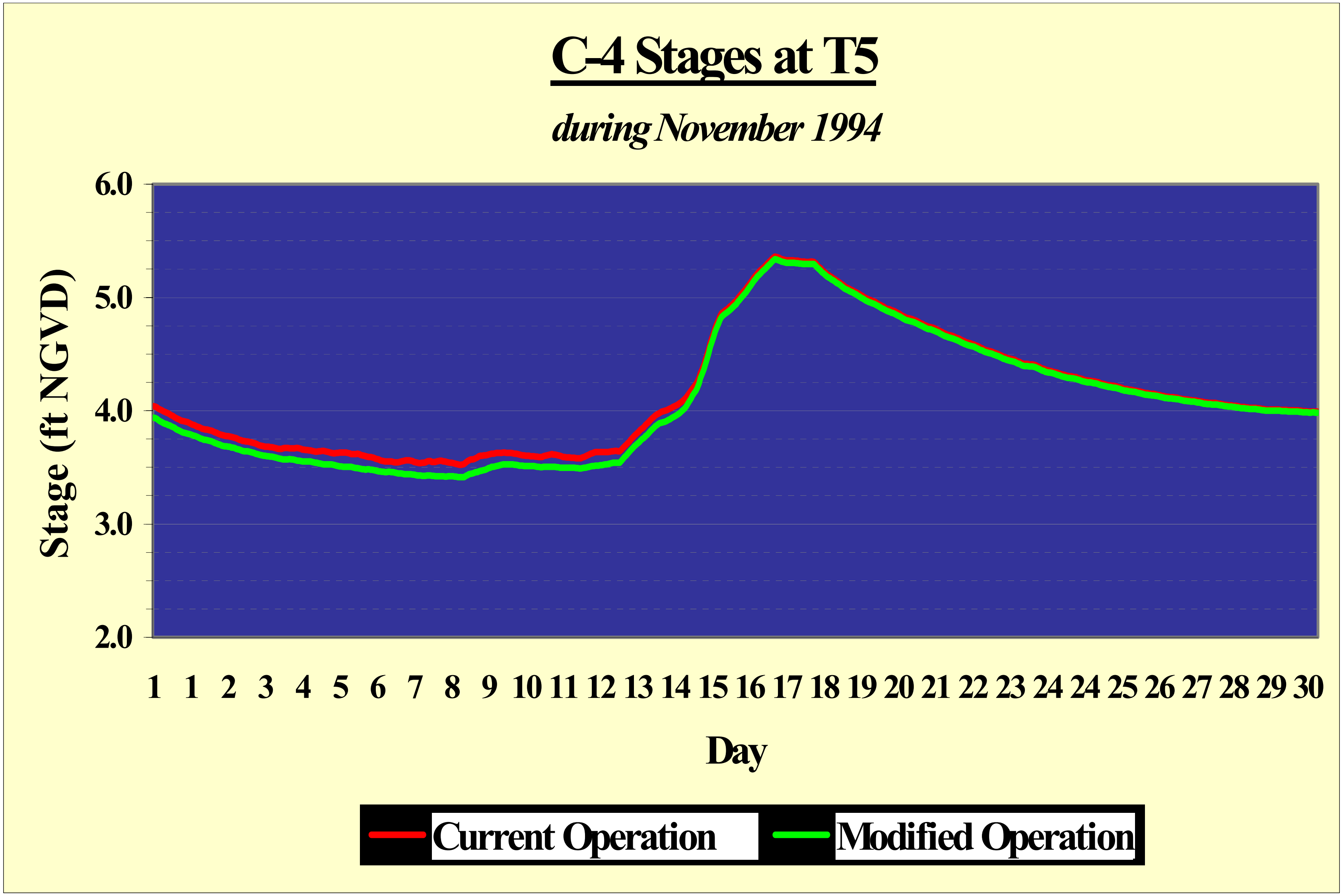


Figure 9. C-4 Stages at T5 Gage for Current and Proposed S-25B Operations

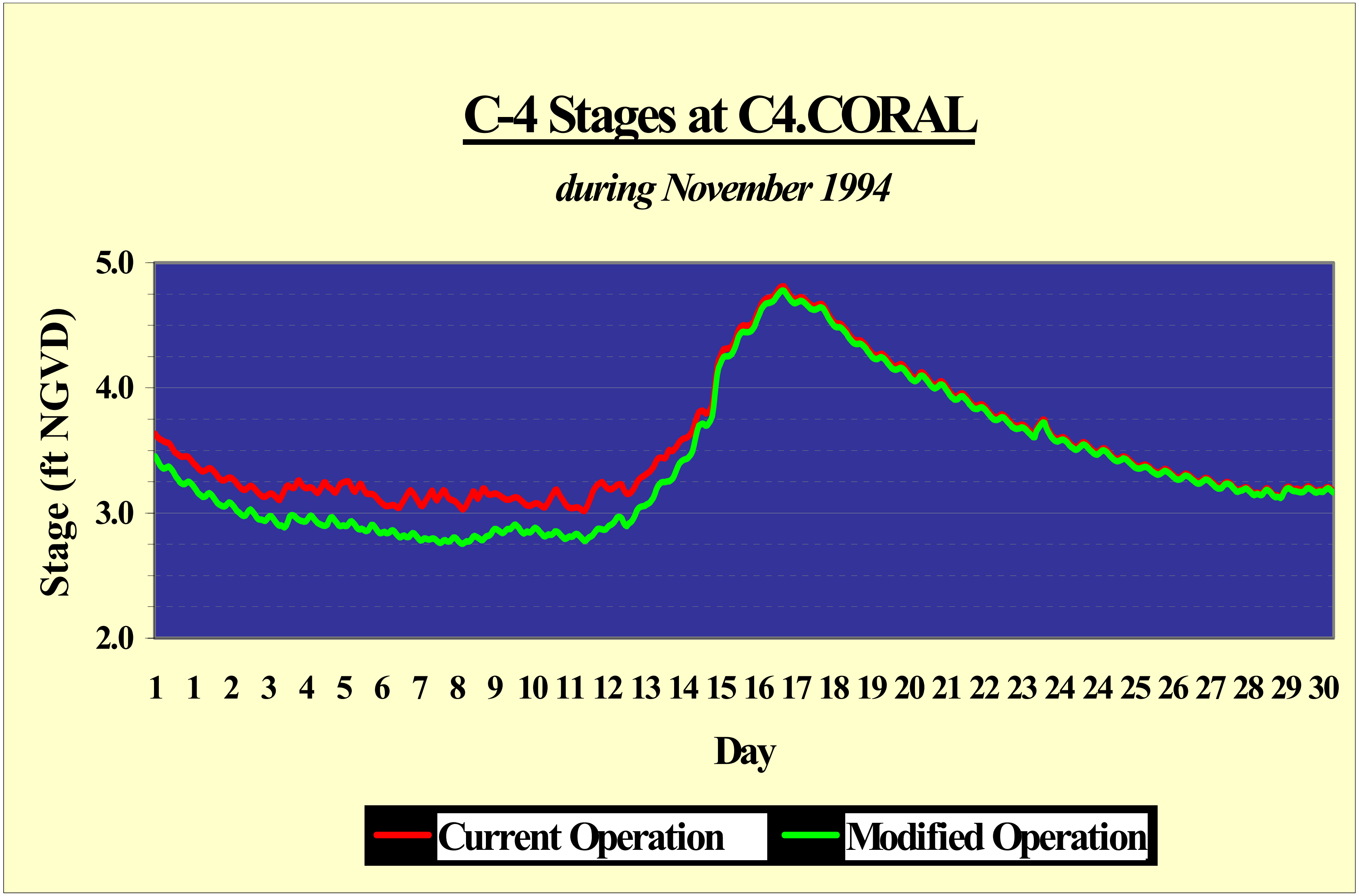


Figure 10. C-4 Stages at C-4.CORAL Gage for Current and Proposed S-25B Operations

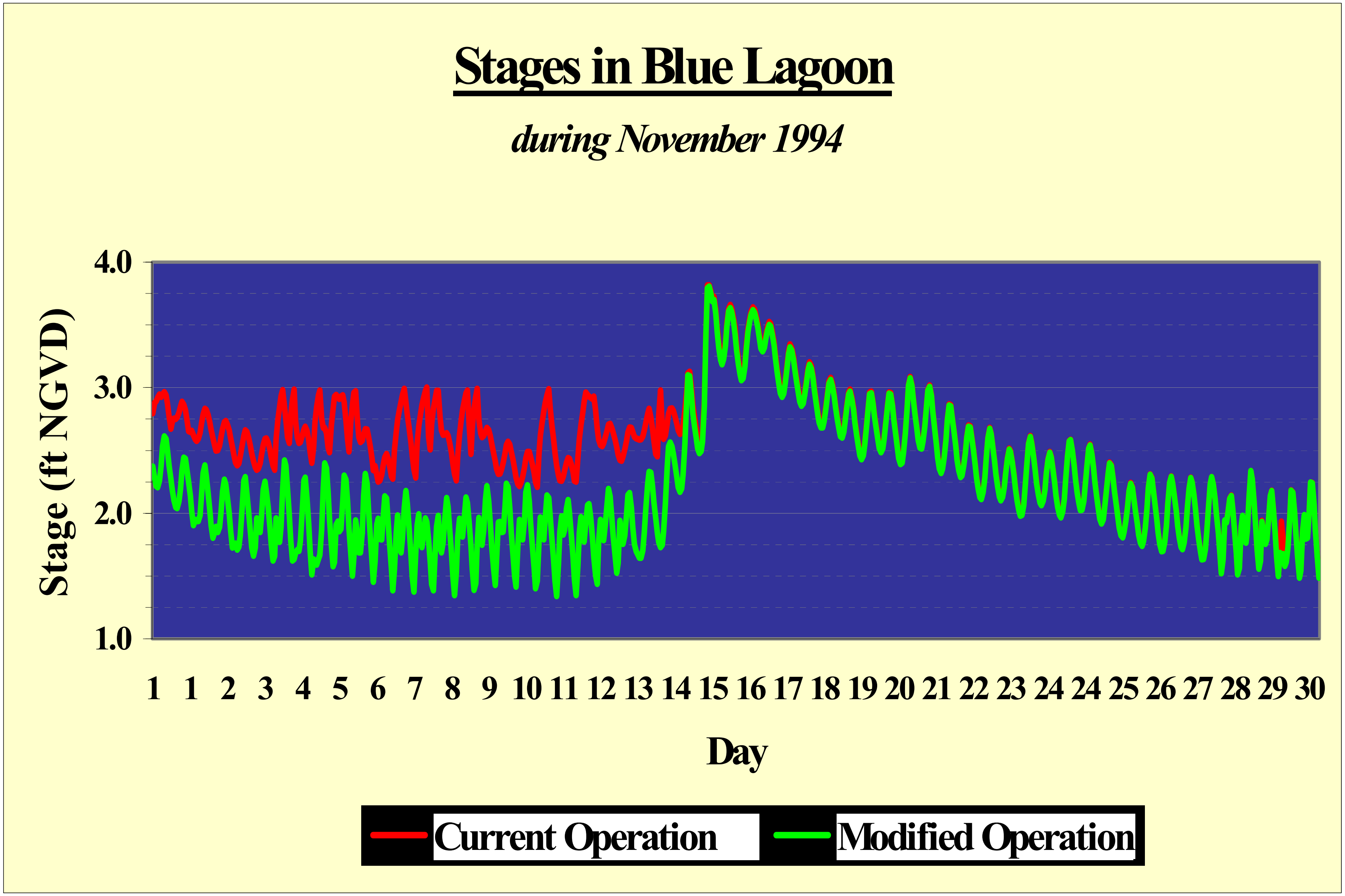


Figure 11. Stages in Blue Lagoon for Current and Proposed S-25B Operations

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## Appendix A. Canal Stage Hydrographs

C-4.CORAL (1)
C-4.CORAL (2)

G-119_T (1)
G-119_T (2)

G-93_H (1)
G-93_H (2)

S-25B_H (1)
S-25B_H (2)

T5 (1)
T5 (2)

## Appendix B. Canal Discharge Hydrographs

C-4.CORAL (1)
C-4.CORAL (2)

G-93_H (1)
G-93_H (2)

S-25B_H (1)
S-25B_H (2)

## Appendix C. Ground Water Level and Wetland Stage Hydrographs

3B-SE_B

G-3328

G-3329

G-3439

G-3559

G-3563

G-3568

G-3570

G-3572

NESRS3_B

## Appendix D. Evaluation of Model Discretization Errors

MODFLOW Model Discretization

The results of Lal (2000) can be used to assess the magnitude of the error $\varepsilon_d$ associated with the spatial resolution. The relationship between cell size, aquifer properties and forcing function is

$$\Delta X = 0.5\sqrt{\frac{T_g \varepsilon_d}{f S_c}} \quad \text{.........................................................} \quad (1)$$

where

$\Delta X$ = the spatial discretization required to simulate a ground water surface profile with $(100 - \varepsilon_d)$ % accuracy (L).

f = the frequency of the disturbance creating the ground water profile ($T^{-1}$).

$T_g$ = the aquifer transmissivity ($L^2$ / T).

$S_c$ = the aquifer storage

Denoting the disturbance period as $P_d$, where $P_d = 2\pi / f$, equation (1) can be rewritten more conveniently as

DRAFT

$$\varepsilon_d = \frac{8\pi S_c \Delta X^2}{T_g P_d} \quad \text{.................................................................} \quad (2)$$

As an example, the disturbance could be caused by the opening and closing of gates at a structure. With $\Delta X = 1000$ feet while assuming $S_c = 0.15$ and 100,000 $ft^2$ / day $\leq T_g \leq$ 1,000,000 $ft^2$ / day, Table D1 shows the values of $\varepsilon_d$ that correspond to various values of $P_d$ and $T_g$. These results illustrate that the percentage error caused by discretization could be quite large where higher frequency disturbances are created within parts of the surficial aquifer system with lower transmissivity. The actual magnitude of the errors would be small, however, at large distances from the disturbance where fluctuations in ground water levels are small. In order to determine how "far reaching" in the aquifer a hypothetical disturbance can be, Lal(2000) demonstrated that the distance at which a one-dimensional

Table D1. %Error, $\varepsilon_d$, at selected values of $P_d$ and $T_g$.

| $P_d$ (days) | $T_g$ ($ft^2$/day) 100,000 | $T_g$ ($ft^2$/day) 1,000,000 |
|---|---|---|
| 0.5 | 75.4 | 7.54 |
| 1.0 | 37.3 | 3.77 |
| 1.5 | 25.1 | 2.51 |
| 2.0 | 18.8 | 1.88 |

disturbance would decay to 5% of the source amplitude can be computed from

$$D_d = 4.3\sqrt{\frac{T_g P_d}{2pS_c}} \quad \text{.........................................................} \quad (3)$$

where $D_d$ is the distance from the source disturbance and which the disturbance amplitude has decayed to only 5% of the amplitude of the source disturbance. Table D2 provides values of $D_d$ for the same values of $T_g$, $S_c$, and $P_d$ given in table D1. Table D2 shows that a disturbance created by structure operations can propagate more than one cell away from the canal. Consequently, the spatial resolution appears to be realistic in regard to this criterion since most of the values in table D2 are greater than 1000 feet. Nonetheless, the accuracy of the computed ground water profile could be improved by increasing the resolution.

As an alternative means for assessing the adequacy of the ground water flow model resolution, an approximate analytic solution by Lal(2001) for evaluating the effects of bank storage on routed channel flows can be applied here to analyze the effects of canal stage perturbations on ground water levels. This analytic solution can be expressed as

Table D2. Values of $D_d$ (ft) for various $P_d$ and $T_g$

| $P_d$ (days) | $T_g$ ($ft^2$/day) 100,000 | 1,000,000 |
|---|---|---|
| 0.5 | 991 | 3132 |
| 1.0 | 1401 | 4430 |
| 1.5 | 1716 | 5426 |
| 2.0 | 1981 | 6265 |

$$H^*(x,y,t) = h'\exp[f\,t + ?x + ?d + \mu(y-d)] \quad (4)$$



where

$H^*$ (L) = the ground water level perturbation.

$h'$ (L) = the amplitude of the canal stage source perturbation, located at x = 0.

f ($T^{-1}$) = $f_1 + f_2 i$; $f_1$ and $f_2$ are the time delay constant and characteristic frequency, respectively, of a stage perturbation with amplitude $h'$ and location at the upstream end of the channel (x = 0).

$\lambda$ ($L^{-1}$) = $\lambda_1 + \lambda_2 i$; $\lambda_1$ and $\lambda_2$ are the decay constant and wave number, respectively, of the disturbances along the channel for x > 0.

$\mu$ ($L^{-1}$) = $\mu_1 + \mu_2 i$; $\mu_1$ and $\mu_2$ are the decay constant and wave number, respectively, of the disturbances within the aquifer ($y \geq \delta$).

$\theta$ ($L^{-1}$) = $\theta_1 + \theta_2 i$; $\theta_1$ and $\theta_2$ are the decay constant and wave number, respectively, of the disturbances within the sediment layer lining the channel ($0 < y < \delta$).

$\delta$ (L) = The effective thickness of the sediment layer.

x (L), y(L) = directions parallel and perpendicular to, respectively, the channel centerline.

t (T) = elapsed time.

Also, it can be shown that

$$\theta\delta = \frac{P_r}{P_m}\Lambda\mu \quad \text{.......................................................} \quad (5)$$

Where $P_r$ is a dimensionless parameter depicting the aquifer and canal, $P_m$ characterizes the canal / aquifer interface, and $\Lambda$ is a characteristic length related to the water level disturbance along the canal. For the C-2 Extension canal, Lal (2001) found that with a perturbation period of 1 day, $P_r = 7.031 \times 10^{-6}$, $P_m = 7.449 \times 10^{-3}$, $\Lambda = 102{,}550$ m and $\mu_1 = \mu_2 = -0.0026\ M^{-1}$. Substituting these values into equation (5) and putting the result along with $f_l = 0$, $x = 0$ and $h' = 1$ foot into equation (4) yields a perturbation magnitude of

$$\left|H^*\right| = \exp\{0.0026[(y-\delta)+96.80]\} \quad \text{.........................................} \quad (6)$$

Table D3. $|H^*|$ versus y - δ for C-2 Extension

| y - δ (ft) | y - δ (m) | $\|H^*\|$ (ft) |
|---|---|---|
| 500 | 152.4 | 0.52 |
| 1000 | 304.8 | 0.35 |
| 1500 | 457.20 | 0.24 |
| 2000 | 609.60 | 0.15 |

Where $H^*$ is in feet. This provides the magnitude of the ground water level perturbation at a distance y - δ from the channel, starting at the perturbation source (x = 0). Table D3 lists values of $|H^*|$ for various y - δ. The values for $|H^*|$ shown in table 3 suggest that for the conditions within the study area, a unit perturbation in canal stage can have significant influence on ground water levels thousands of feet away from the canal. While this seems to indicate that the selected resolution of 1000 feet is adequate, the results in table D3 also illustrate that a finer resolution could lead to greater accuracy in the computed ground water levels. Future versions of this model should consider this while the current version should only be used for preliminary comparisons of different scenarios for flood management within the C-4 basin.

Numerical errors in computed water levels can also arise from discretization of the time domain. Lal (2000) indicated that the time step Δt needed to limit errors to 5% of the disturbance amplitude can be computed from

$$\Delta t = \frac{0.14\Delta X^2 S_c}{T_g} \quad \text{................................................................} \quad (7)$$

With $\Delta X$ = 1000 feet and $S_c$ = 0.15, the time step sizes associated with transmissivity values of 100,000, 500,000 and 1000,000 $ft^2$/day are 5, 1 and 0.5 hours, respectively. A MODFLOW time step of 1 hour was used in the simulations.

UNET Model Discretization

Channel cross sections were specified in the UNET model at each location where

- a canal crosses a MODFLOW cell boundary;
- significant changes in cross sectional geometry occur;
- a canal reach is interrupted by an internal boundary;
- the end of a UNET reach occurs; and
- a monitoring station exists.

Additional computational nodes (i.e. interpolated cross sections) were inserted so as to limit the computational node spacing to 500 feet.

Given a spatial discretization, the Courant condition can be used to determine the time step needed to maximize the accuracy of the finite difference scheme. This condition can be expressed as (Fread 1974)

$$\Delta t \leq \frac{\Delta x}{V + \sqrt{gh}}\left[1 - 0.452gn^2|V|\Delta t R^{-4/3}\right] \quad \text{........................................} \quad (8)$$



Table D4. $\Delta t_{max}$ (minutes) versus $\Delta x$ (ft) for C-4

| $\Delta t_{max}$ (minutes) | $\Delta x$ (ft) |
|---|---|
| 0.078 | 100 |
| 0.24 | 500 |
| 0.32 | 1000 |

For the C-4 canal, representative values of V, h, n, and R would be 2 ft/s, 8 ft, 0.1 and 5 ft, respectively. Substituting these values into (8) with $\Delta x$ = 100, 500 and 1000 feet yields the maximum values for $\Delta t$ shown in table 4. These results show that the Courant criterion is fairly restrictive for the conditions encountered within the study area. However, Barkau(2001) indicates that, in practice, the use of a larger time step often results in a negligible decrease in accuracy. The effects of time step size on model results can be investigated through sensitivity analysis.

# Appendix E. Sensitivity of C-4 Stages and Flows to Selected Model Parameters

| Parameter | Stage Sensitivity | | | | Discharge Sensitivity | | | |
|---|---|---|---|---|---|---|---|---|
| UNET $\Delta t$ | **D-B Levee JCT** | **T5** | C4.CORAL | **T3** | **D-B Levee JCT** | **T5** | C4.CORAL | **T3** |
| UNET $\Delta x_{max}$ | **D-B Levee JCT** | **T5** | C4.CORAL | **T3** | **D-B Levee JCT** | **T5** | C4.CORAL | **T3** |
| Aquifer K | **D-B Levee JCT** | **T5** | C4.CORAL | **T3** | **D-B Levee JCT** | **T5** | C4.CORAL | **T3** |
| Reach Trans. | **D-B Levee JCT** | **T5** | C4.CORAL | **T3** | **D-B Levee JCT** | **T5** | C4.CORAL | **T3** |
| Rainfall Recharge | **D-B Levee JCT** | **T5** | C4.CORAL | **T3** | **D-B Levee JCT** | **T5** | C4.CORAL | **T3** |

## Appendix F. Additional Analysis of a Constricted C-4 Reach

The surveyed cross sectional data acquired from the C-4 canal from Barnes (2002) indicate that an unusually shallow cross section exists within the vicinity of station 276+31, although its extent along the canal centerline was not determined. Consequently, HGL profiles computed by the model show a significant head loss within the canal reach located between the C-3 junction and the next canal downstream (**figure 2**). This appendix contains the results of an additional data analysis that was conducted to see if measured stages and flows were consistent with model results within the subject reach. Included also are selected results of the three operational scenarios resimulated with the constricting cross section removed from the model.

Stage and Flow Data

The closest monitoring stations located upstream and downstream of the constriction are C4.CORAL and T3, respectively (**figure 5**). C4.CORAL has continuous flow and stage data while the only data acquired at T3 are daily stage observations. Flows into the reach were approximated by subtracting flows through G-93 from discharges at C4.CORAL. Also, mean daily stages and flows were used in order to minimize scatter in the data. Head losses across the reach were estimated using stage differences between the two monitoring site locations. Since mean daily stages were not available at the T3 site, the actual daily water readings were used. Figure F1 portrays the measured mean daily discharge plotted against the measured head loss. Shown also for reference are hypothetical discharges that are based on steady uniform flow conditions for each measured head differential. The upper curve reflects a channel conveyance of 50,000 CFS and is representative of unconstricted cross sections for the reach. Similarly, the lower curve depicts the conveyance of the constricted cross section (approximately 11,000 CFS). Both conveyance values are based on a typical stage value of 3.0 feet. Consequently, the measured mean daily flows shown in figure F1 were limited to those corresponding to a mean daily stage at C4.CORAL between 2.5 and 3.5 feet.



Analytic Evaluation

Figure F2 portrays a simplified conceptualization of the C-4 canal reach under consideration. The constriction is represented by a broad-crested weir with a trapezoidal cross section. Head losses incurred within the channel itself, both upstream and downstream of the weir, were computed with Manning's equation. Upstream of the weir, this amounts to

$$H_{US} = H_{CORAL} - L_{US}\frac{Q_S^2}{C_V^2} \qquad \text{(F1)}$$

where $H_{US}$, $H_{CORAL}$, $L_{US}$, and $Q_S$ are as shown and $C_V$ is the channel conveyance. $H_{CORAL}$, the stage at the upstream end, was set at 3 feet. Applying Manning's equation to the channel downstream of the weir results in

$$H_{T3} = H_{DS} - L_{DS}\frac{Q_S^2}{C_V^2} \qquad \text{(F2)}$$

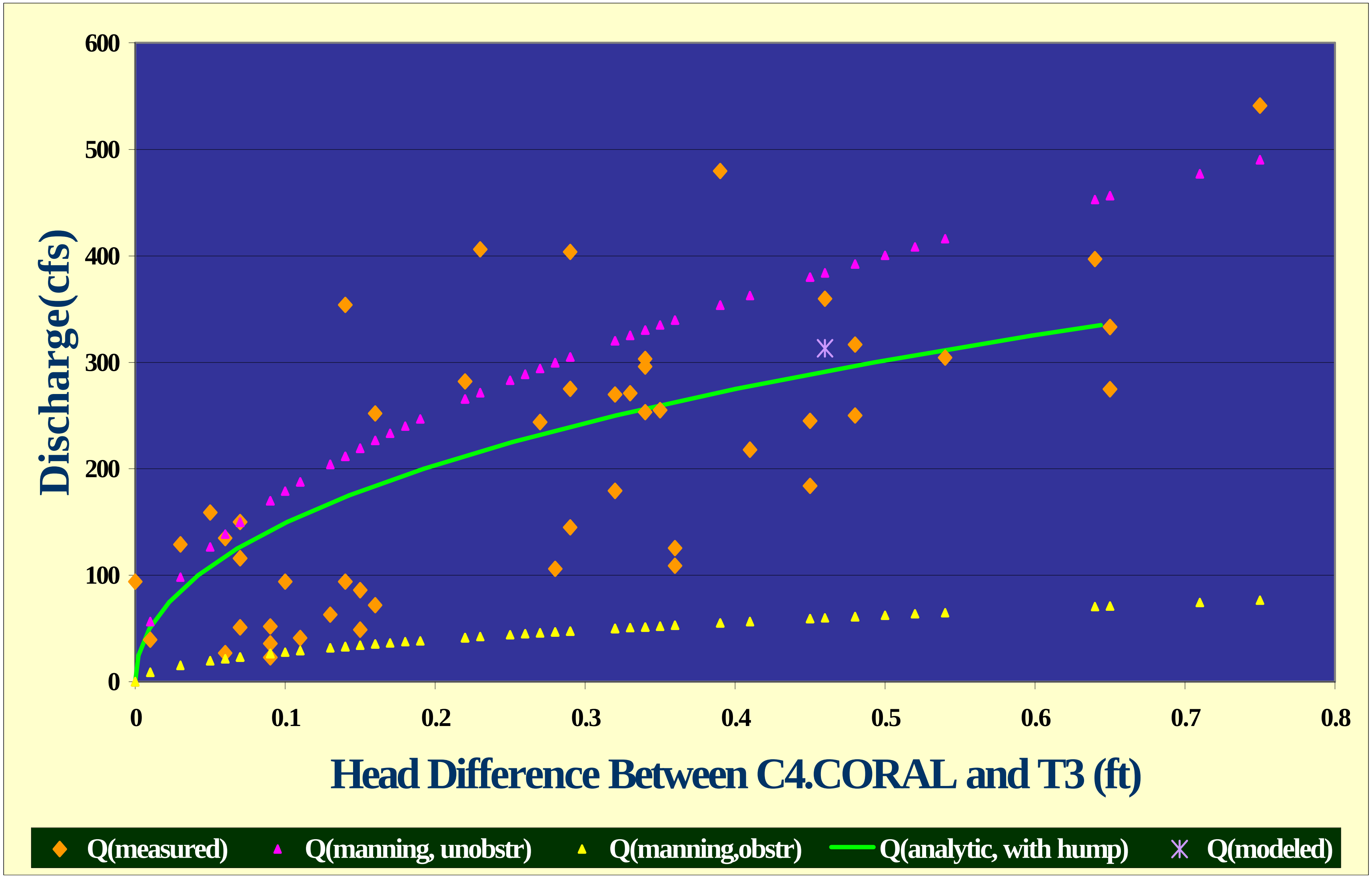


Figure F1. Discharge Versus Head Difference for an Obstructed Reach in C-4

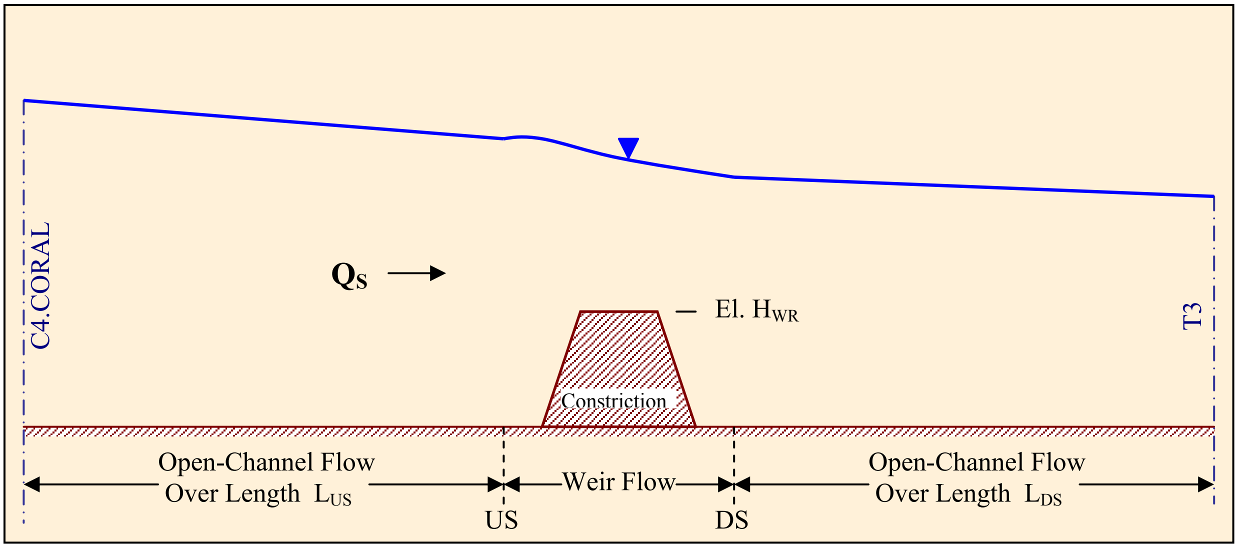


Figure F2. Simplified Conceptualization of C-4 Reach with Constriction

Although the application of equations (F1) and (F2) neglects backwater effects, the resultant stages can still provide some idea of the amount of head lost within the unconstricted portion of the channel. The head loss across the constriction conceptualized by the weir can be estimated by

$$Q_{UNS} = CL(H_{US} - H_{WR})^{3/2} \quad \text{(F3)}$$

$$Q_S = Q_{UNS}\left[1 - SF_{WEIR}^{3/2}\right]^{0.385} \quad \text{(F4)}$$

$$SF_{WEIR} = \frac{H_{DS} - H_{WR}}{H_{US} - H_{WR}} \quad \text{(F5)}$$

where C is the weir coefficient and L is the weir width. Equation (F3) is used to compute the hypothetical unsubmerged discharge across the weir ($Q_{UNS}$) while equations (F4) and (F5) can be used to compute the actual submerged discharge ($Q_S$). These latter two formulations constitute the Villamonte equation. Combining equations (F3) through (F5) results in

$$H_{DS} = (H_{US} - H_{WR})(1 - R_Q^{2.597})^{2/3} + H_{WR} \quad \text{(F6)}$$

where $R_Q$ is the ratio of submerged to unsubmerged flow. Stage differences between C4.CORAL and T3 were estimated using equations (F1), (F2) and (F6) along with assumed values of discharge. Values for L, $H_{WR}$, $L_{US}$ and $L_{DS}$ were estimated from geometric data. Furthermore, according to the Brater and King (1976) Handbook of Hydraulics, Tables 5-9 and 5-10, C ≈ 3.5. The resulting analytical relationship between discharge and head loss is shown in figure F1. Also shown for comparative purposes is the value of discharge versus head loss computed in the simulation of wet season equilibrium conditions.

Modified Model Simulations

In order to test the sensitivity of the three model scenarios to the alleged obstruction, all three simulations were redone with the constricting cross section removed. Figures F3a and F3b depict how the results shown in figures **6b** and **6d**, respectively are impacted. Additionally, figure **F4**

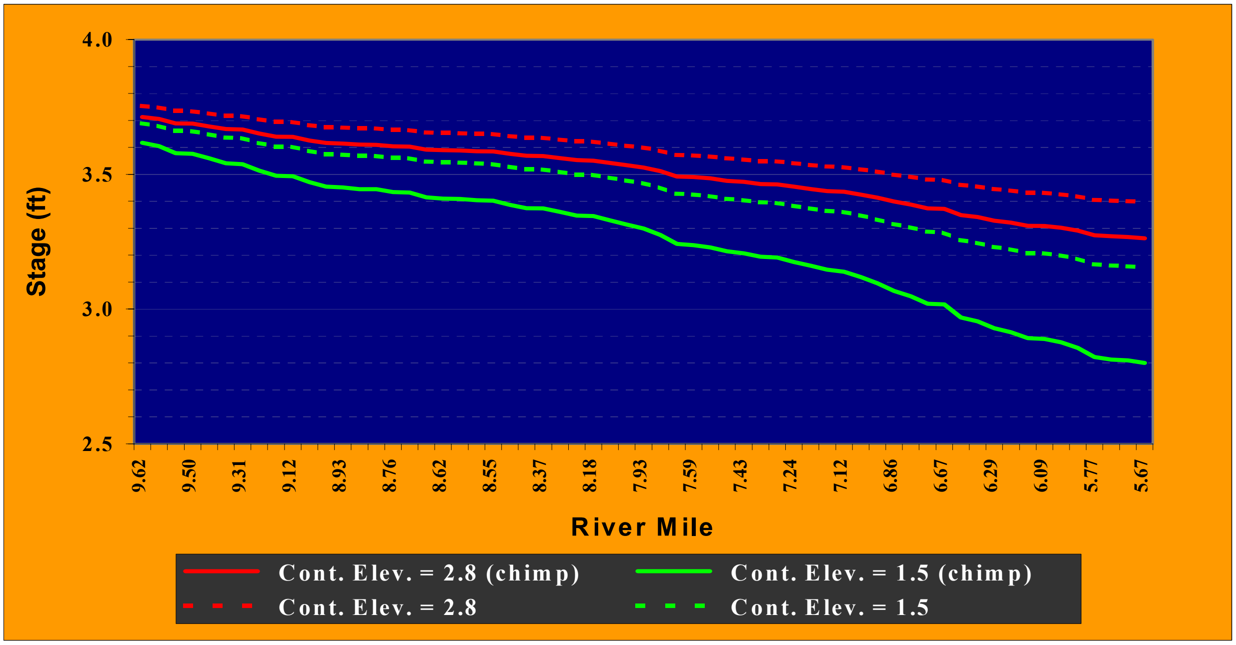


Figure F3a. Comparison of C-4 Wet Season Stage Profiles With and Without Constriction



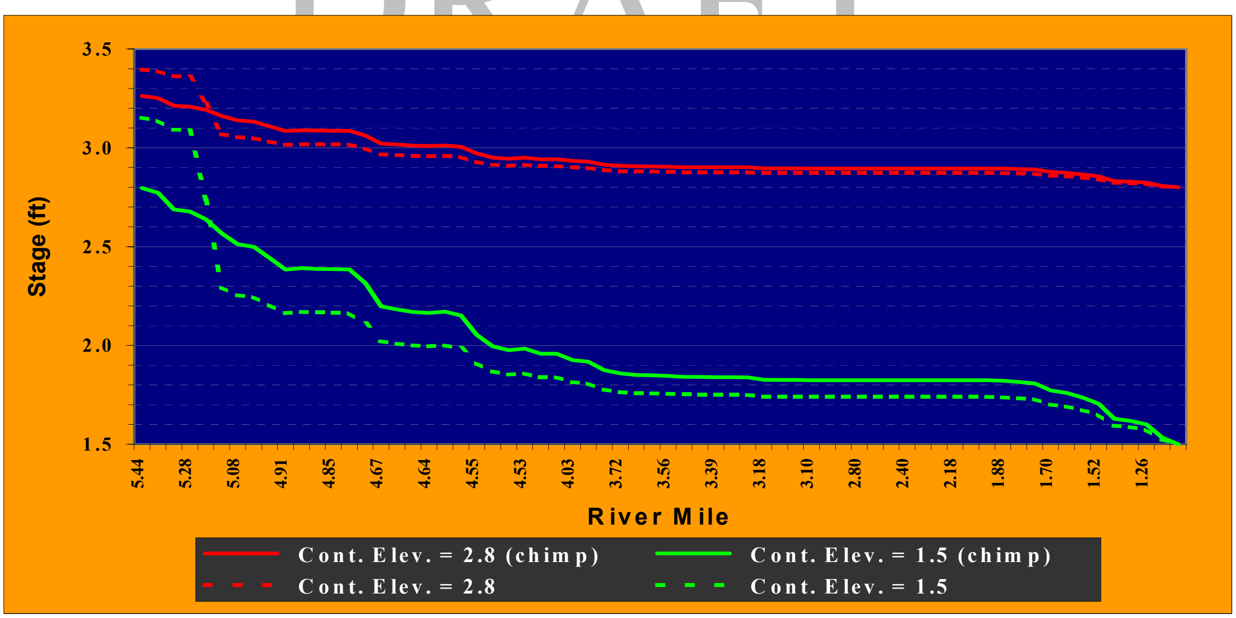


Figure F3b. Comparison of C-4 Wet Season Stage Profiles With and Without Constriction (con't)

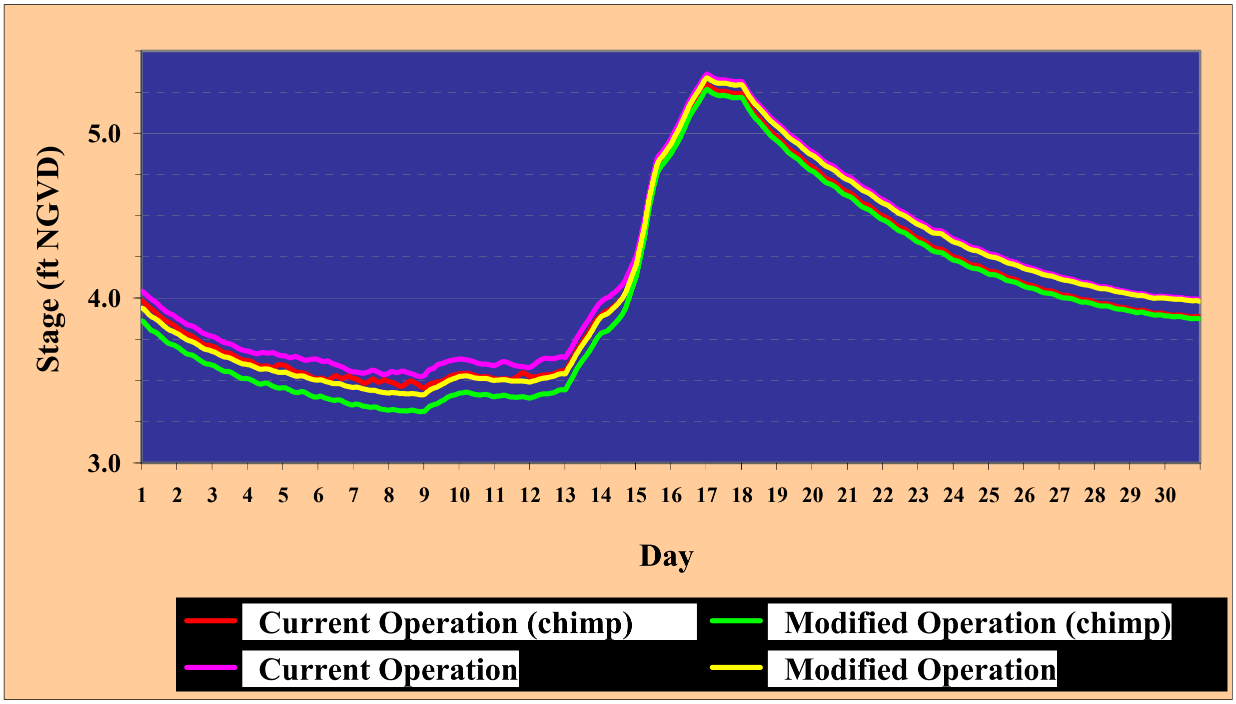


Figure F4. Comparison of T5 Stage Hydrographs With and Without Constriction

contains the results shown in **figure 9** along with the revised stage hydrographs reflecting constriction removal. In each figure, the acronym "chimp" denotes "channel improvement (i.e. the constriction was removed). It is evident in **figure f3a** that removing the obstruction can increase the drawdown in canal stage from about 0.1 foot to about 0.3 foot near the downstream end of Sweetwater. Similarly, the drawdown increases from about 0.25 foot to about 0.6 foot near the C-3 junction. Furthermore, figure F4 suggests that stages at T5 would be about 0.1 foot lower after the peak of the storm event.



Discussion

While the relationship between discharge through the problem reach and head loss across it is somewhat sketchy due to the scatter in the data, the uncertainty in the various parameters used in equations (F1) through (F6) and the simplistic nature of this analysis, the measured data do seem to exhibit general agreement with the conceptualization portrayed in figure F2. Alternatively, given the conceptual model used in this analysis along with the estimated values for channel conveyance, the data generally depict a greater total head loss across the reach than can be explained by friction losses alone. Consequently, this analysis is consistent with the model results and supports the recommendation that additional data be acquired to better define the geometric properties of this canal reach. Also, if such an obstruction exists as it is depicted in the model, a salient (but still somewhat small) improvement in flood reduction benefits may be realized at Sweetwater.